PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
MAY 1952

ORDINARY MEETING

22 January, 1952

ALLAN STEPHEN QUARTERMAINE, C.B.E., M.C., B.Sc.,
President, in the Chair

The President said that the Council recommended to the members present the election of His Royal Highness the Duke of Edinburgh, K.G., R.N., as an Honorary Member of the Institution, because of his distinguished position and of the interest His Royal Highness had evinced in the promotion of science and in particular its application for the use and benefit of mankind.

The recommendation was adopted with acclamation.

The Council reported that they had recently transferred to the class of

Members

Dennis Percy Bertlin, M.Eng. (*Liverpool*).
Stanley Baines Hamilton, Ph.D., M.Sc. (*Lond.*), B.Sc. (Eng.) (*Lond.*).
Albert James Hill, B.Sc. (Eng.) (*Lond.*).
Denis Rebbeck, C.B.E., M.A. (*Cantab.*), M.A. (*Dublin*), M.Sc. (*Belfast*), Ph.D., B. Litt. (*Dublin*).
Walter Douglas Sangwin.
Victor Henry Tripp, B.Sc. (Eng.) (*Lond.*).

and had admitted as

Students

James Charles Christopher Alford.
Norman Edward Allender, B.Sc. (Eng.) (*Lond.*).
Patrick James Anderson.
Anthony Bailey.
Henry Findlay Baird.
Richard Brian Bavister.
Derrick Beckett.
Derek George Bellamy.
Amer Singh Bhatt.
Derek George Bird.
Alan James Briscoe.

William Edward Wilby Brook.
George Frederick Brown.
John Neville Bulman.
John Herbert Caiger.
Gordon Ross Cameron.
John Dall Chalmers.
Binoy Kumar Chatterjee, B.Sc. (*Patna*).
Biswanath Chatterjee.
Peter Geoffrey Clark, B.A. (*Cantab.*).
John Loraine Cobb.
John d'Arcy Conway.
John Michael Cornish.
John Cowan.
Antony Caton Crozier, B.Eng. (*Liverpool*).
Christopher Ernest Davison.
Kenneth William Day.
Anthony Francis Joseph Dias.
David Dick.
Peter John Dimery.
Gordon Arthur Dobbs.
John Ellington.
Stephen Runciman Farrell.
Ebenezer Olawanle Oladipo Fasehun.
Thomas White Finlay.
Michael Edward Kirk Foulkes.
Roy Morton Foulsham.
William George Gadd.
Michael Robert Gander.
Thomas Thomson Gardner.
William Arthur Green.
Adam Dunsmuir Hall.
Edwin Oliver Hamblen-Thomas, B.A. (*Cantab.*).
Ian Fleming Hamilton.
Frederick William Hawks.
Robert Mervyn Hearst.
Derek John Hillier, B.Sc. (Eng.) (*Lond.*).
Leslie Norman Barry Holden.
Geoffrey Crosland Holmes.
George Edward Hone.
John Howard.
Edward Victor Jenkins.
Eric Graely Jenkins, B.Sc. (*Wales*).
Hywel Glyndwr Hamer-Jones.
John Charles Jones.
Wyndham Charles Jones.
Dennis Kentish.
William Rodney Stewart King.
Lee Ting-Wai, B.Sc. (*Hong Kong*).
James Lees.
Geoffrey Hesketh Lewis.
Charles Augustus Liburd.
Peter Michael Ling.
Alec Martin Luxon.
John McLachlan McCall.
Malcolm Dominic Macaskill.
Thomas John Andrew McAuley.
Malcolm Allan MacDonald.
Arthur Ronald McFarland.
Ian Osborne Mackenzie.
Andrew Stevenson McLean.
Thomas Lyle McNab.
Robert Begg McNee.
Duncan MacPhail.
Raymond Peter Manning-Coe.
Louis Robespierre Marais.
Ronald Victor Martin.
William Martin.
John Julian Alastair Maw.
Colin Mayfield.
Richard James Meader.
George Mercer.
Ronald William Mill.
Edwin Paul Miller.
James Christopher Moore.
Horace Llewellyn Morancie.
John Charles Spencer Mott.
Terence James Mullett.
David Cope Newmark.
Raymond Sydney George Nicol.
Stanley Navaratnam Niles.
Ian Crawford Nish.
Philip Alan Northfield.
Edwin George Page.
John David Parr-Burman.
Charles Peter Pattullo.
Rudolf Theodor Felix Plaut.
George David Rattue.
John Edward Rickelton.
Ralph Berry Sims.
Baldev Singh-Sohi.
David Ian Small.
Alan Gibson Smith.
William Christopher Sprott.
Barry Wootton Staynes.
Joseph Stewart.
John Adam Taylor.
Anthony Brian Stafford Tidy.
Leslie Frank Titterrell.
Philip Toman.
Michael Fernihough Tong.
Robert Martin Topp, B.A. (*Dublin*).
Antonie van Heerden, B.Sc. (*Cape Town*).
Gordon Waltham.
Michael John Wilde.
Yap, Chin Leong.
Alan Barrington Yates.
Gerald James Yeend.

The Scrutineers reported that the following had been duly elected as

Associate Members

ROY PENNINGTON ASHTON, B.Eng. (*Liverpool*).
JOSEPH CHARLES BEACHUS.
DANIEL FLETCHER BINNION, B.Sc. (*Manchester*), Stud. I.C.E.
GORDON ARNISON CAMPBELL, M.Sc. (*Leeds*), Stud. I.C.E.
FRANK CARTWRIGHT, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
EDWARD ALEXANDER COMERFORD, B.A., B.A.I. (*Dublin*).
TOM CONSTANTINE, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
COLIN CAMPBELL CRAIG, Stud. I.C.E.
THOMAS FINLAYSON CRAIG, Stud. I.C.E.
JAMES ANTHONY DUNSTER, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
JOHN RICHARD FLETCHER, B.Sc. (*Wales*).
HORACE WILLIAM ALEXANDER FRANCIS, Stud. I.C.E.
PETER HARLE GILES, Stud. I.C.E.
ADOLF GOLDSTEIN, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
WILLIAM THOMAS GREER, B.Sc. (*Glas.*), Stud. I.C.E.
PERCY PHILIP GRIFFITHS, B.Sc. (*Wales*), Stud. I.C.E.
BERNARD JEAN D'ARCY HARLOW, B.Sc. (*Leeds*).
WILLIAM JOHN HASKELL-THOMAS, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
JOHN FULLER HOLMAN, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
PETER WILLIAM JAMES, B.Sc. (*Durham*), Stud. I.C.E.
RICHARD DENIS JENNINGS, B.Sc. (Eng.) (*Lond.*).
JOHN ALISTAIR LANGBEIN, B.E. (*New Zealand*), Stud. I.C.E.
KENNETH HENRY LEE, B.Sc. (Eng.) (*Lond.*).
DONALD MCKENZIE MILLER, Stud. I.C.E.
RAYMOND MITCHELL.
JOHN FREDERICK HERSE CROFTS OTTLEY, B.Sc. (*Leeds*).
MAXWELL CHARLES PURBRICK, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
EL ZEIN SACHAYROON.
ALAN THOMPSON STUCKEY, B.Sc. (Eng.) (*Lond.*).
JOHN AUBREY TINGLE.
GEORGE WEBSTER, B.Sc. (*Durham*), Stud. I.C.E.

The following Paper was presented for discussion and, on the motion of the President, the thanks of the Institution were accorded to the Author.

Paper No. 5851

"Civil Engineering Aspects of Hydro-Electric Development in Scotland"

by

Angus Anderson Fulton, B.Sc., M.I.C.E.

SYNOPSIS

After reviewing the many factors such as topography, climate, geology, existing uses of rivers, constructional methods, destination and terms of sale of output, active opposition, etc., which influence the design of hydro-electric developments, the Paper deals with the relative importance of these factors in Scotland.

How Scottish designs are affected by such considerations as seasonal variations in rainfall and flow, load characteristics, site limitations, plant efficiency, and material and labour shortages are discussed in detail. Examples are quoted of the types of structures and equipment which are being applied in Scotland for the impounding and control of water and for the ascent of fish.

After dealing with the selection, assessment, promotion, and construction of schemes, a picture is given of the stage reached and the apparent limit of development in Scotland. This is followed by a description in some detail of the essential engineering features of the principal schemes—completed and in progress—with appropriate explanations for the forms which the developments and the designs have taken in each case.

Introduction

There is generally some recognizable engineering feature or characteristic which identifies a hydro-electric development with a particular country. It is usually quite obvious. For example, extremely-high-head installations are found only in very mountainous countries such as Switzerland and Norway. Or again, large barrages and headrace canals are linked with the Rhine or some other large continental rivers. It is not quite so easy to particularize in the case of Scotland, which possesses neither high heads nor large rivers. However, schemes employing elaborate networks of collecting aqueducts may be said to be essentially Scottish. Of course, not all schemes in Scotland follow this pattern, but a small country with limited water-power resources must turn every available stream to account and the water-power development map of Scotland shows how few catchments will not make a contribution to power production. Apart from the economic need there are several other influences which have had an effect on the form of water-power development in Scotland.

Briefly, these include topography, climate, seasonal variation in rainfall, character of water, geology, availability of constructional materials, existing uses of land and water-courses, presence of fish, method of carrying out the work, terms of sale of output, nature of opposition to development, and defence considerations.

Natural and Physical Conditions

The topography of the Highlands has, on the whole, helped hydro-electric development in Scotland. It provides an elevated moorland of over 3,000 feet above O.D. on the west, falling gradually towards the east, and cut by deep valleys. Many of these valleys fall sufficiently slowly to offer good storage basins, but where formed by glacial action they tend to be wide and open with much overlying drift, which is not ideal for dam sites.

The climate also is favourable for water-power. Temperatures are moderate and evaporation losses small and reasonably constant from year to year. Rainfall is even, with more in the winter than in the summer months; the difference is increased (in terms of run-off) by greater evaporation during the summer, as shown in Table 1. (The ratio is very nearly 2 : 1.)

Table 1.—monthly distribution of rainfall and run-off

	Rainfall: % of annual		Run-off: % of annual	
October . . .	10·5		11·0	
November . . .	10·5		11·5	
December . . .	12·5		13·5	
January . . .	11·0		12·0	
February . . .	9·5		10·0	
March	7·0	61·0	7·5	65·5
April	8·5		8·5	
May	4·0		3·5	
June	5·0		3·5	
July	5·5		4·0	
August	7·0		6·0	
September . . .	9·0	39·0	9·0	34·5
		100%		100%

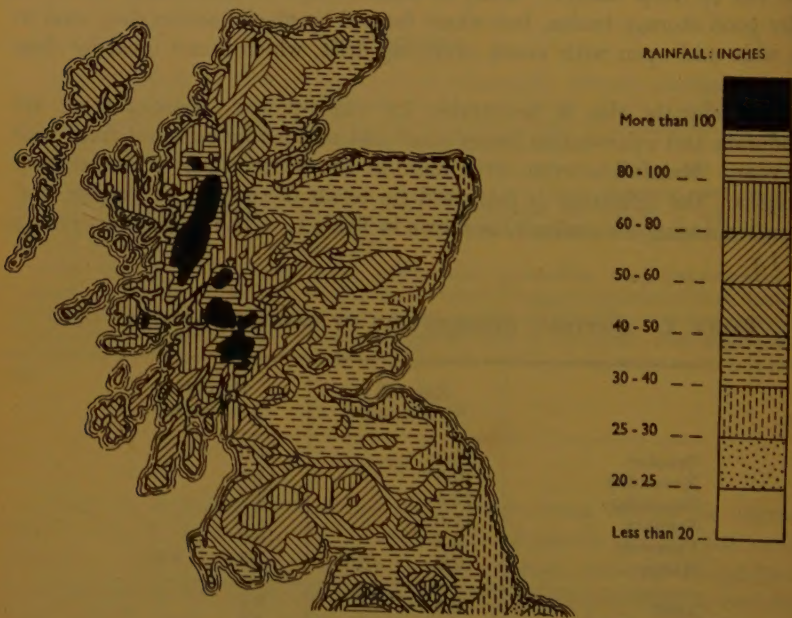
The intensity of rainfall ranges from considerably more than 120 inches in the high wet areas of the west to as little as 25 inches per annum in the east, with a corresponding influence on location of developments. This is illustrated by *Fig. 1*. The humid prevailing south-west winds moderate the influence of winter and, apart from isolated and outstandingly cold

spells, virtually ensure an immediate availability of the winter's run-off. This is allowed for in deciding generating-plant capacity and has an important influence in planning the storage to be provided for power production.

The annual diversity of rainfall in Scotland is considerable but not crippling. For areas of reasonable size it varies from about 70 per cent of the long-term average for a dry year to about 150 per cent in a wet year (see *Fig. 2*). The average of 3 successive dry years is about 80 per cent of the long-term average.

Such large differences between a dry year and a wet one make it

Fig. 1



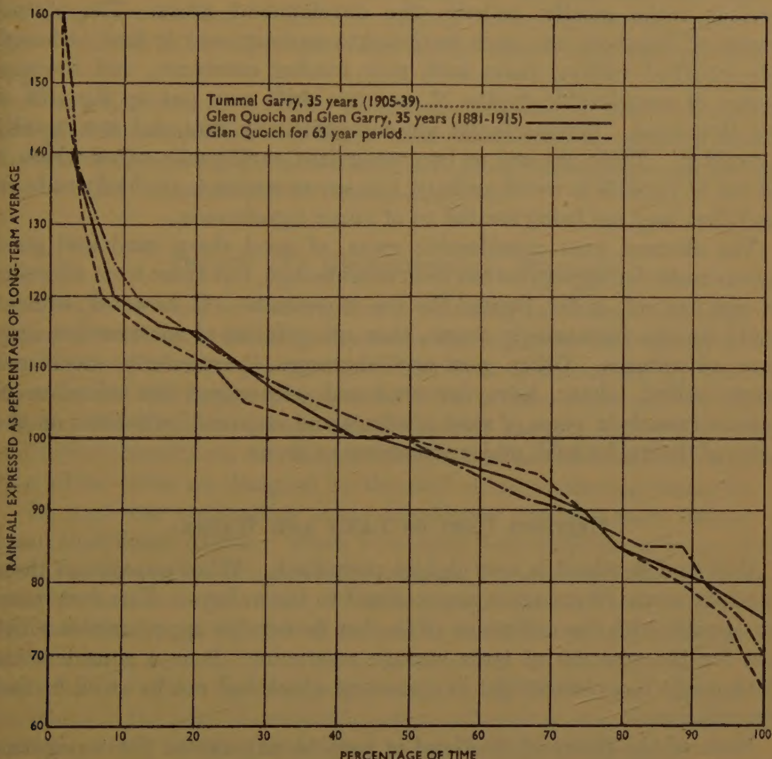
ANNUAL RAINFALL

almost impossible to provide, at reasonable cost, storage large enough to average out dry and wet years. Instead the aim is to provide reservoirs large enough to average out seasonal variations at least and to ensure that the average flow of a dry year is always available. Table 2 shows the flow availability for different capacities of reservoirs as given by W. J. E. Binnie.[1]

Where, however, the water is used to generate electricity for public supply, the results are better than shown because the demand eases off

[1] W. T. Halcrow, "The Lochaber Water-Power Scheme." Min. Proc. Instn Civ. Engrs, vol. 231 (1930–31, Pt 1), p. 31, *Discussion* by W. J. E. Binnie, p. 71.

Fig. 2



ANNUAL DIVERSITY OF RAINFALL

TABLE 2

Storage : per cent of long-term average flow	Guaranteed Flow : per cent of long-term average flow
15	57
20	66
25	71
30	75
35	77

when the natural run-off is at its lowest. For storages in the range 20–30 per cent of the average flow, an improvement of about 5 per cent in the guaranteed flow is likely on this account. For the same range of storages, the available discharge in an average year (provided there is adequate plant capacity) may be about 15 per cent more than the figures given in Table 2.

Geologically Scotland is suitable for waterpower development. Impervious rocks mostly underlie the development areas. The virtual absence of limestone has made watertight reservoirs easy to find, although wide-mouthed valleys, rocks with poor bearing resistance, and the prevalence of considerable depths of morainic drift may put up the cost of dam structures. Serious faults are comparatively rare and can usually be avoided. There are one or two recognized earthquake zones which it will not be possible to avoid entirely, but any movements are likely to be so small that they are being treated as of minor significance.

The absence, over considerable areas, of good sharp sand and good igneous rocks for aggregates has been troublesome, but apart from affecting the cost has not, so far, limited the use of concrete. If, however, cement should become increasingly scarce, then other forms of construction may show advantages. Other post-war shortages, particularly steel and certain skilled labour, have favoured and encouraged the adoption of pressure tunnels in place of steel pipelines and imposed limitations on the choice of design for such major structures as dams.

Existing Uses of Land and Water

Highland Scotland is very lightly populated. What population there is and its means of existence are confined to the valleys. This fact interferes greatly with the utilization of the few favourable opportunities which exist for the creation of large storage reservoirs. It is a consideration which might have less weight in a country which had not to avoid further depopulation.

None of the rivers of Scotland is used to any extent for navigation and not at all for timber transportation, but most carry migratory fish. The presence of fish and the need to maintain the stock is profoundly influencing all hydro-electric development. The provision of passes to allow salmon to ascend and descend, and of screens to prevent them entering the works, has complicated design work and necessitated a high level of compensation water where stretches of rivers are by-passed. Scotland has no scheme where a major river has been diverted into the watershed of another or into the sea at a point well removed from its natural outlet. The reason is that the lowest power station must discharge its water inside the river mouth if fish seeking that river are to have sufficient attraction to take them into it.

One of the provisions made in the interests of the continuance of fish-life in Scottish rivers, namely compensation flows, also prevents any complaints about such spoliation of existing scenery as dried-up river beds. Regard for amenity is also seen in the attention paid to architectural features of all main structures, such as dams, power stations, and control buildings. Restrictions have been accepted too in the amounts by which reservoir water-levels may be varied. Such limitations to the flexible use

of available storage is of more consequence in Scotland than in a country like (say) Switzerland, which also applies similar limitations, but differs from Scotland by having ample water in the summer when such restrictions apply.

The objections to pipelines have been removed in those cases where, owing to favourable factors, it has been possible to use pressure tunnels. In some instances the power stations have gone underground too, but this has been for economic and not defence reasons.

Design and Constructional Methods

Most hydro-electric construction in Scotland has been undertaken by engaging consultants to design the works and supervise the contractors who are employed to carry them out. When work is carried out by contract, there is less freedom for changes after it starts than is the case with direct labour. On the other hand, where there is enough employment for a number of consultants, stereotyped designs and pet ideas are less common than when works are designed by the staff of the employing authority.

In post-war contracting some significant changes have tended to upset traditional habits. When a contract was undertaken in pre-1939 days, the contractor automatically assumed responsibility for all changes in labour and material rates. If these changes were adverse, or if he met any bad luck, it was his job to cope with the situation and, by ingenuity and economy, he usually contrived to come out on the right side. Such characteristic efforts by contractors would avail little today, because the variations in the price of everything are so substantial that few contractors could hope fully to offset their effect by any action on their part.

The urgency with which hydro-electric construction has proceeded since the war in order to try and keep pace with the growth of load has not given the designers of the Scottish works many opportunities to strike out on new lines. There are consequently fewer examples of novel civil engineering design in Scotland than in other countries. With more time for investigation, Sweden might not have been alone in experimenting with very thin reinforced-concrete curtain walls in rockfill dams and in specializing in underground power stations with long tailrace tunnels. Scotland has not the narrow gorges which have given France the opportunity of developing ski-jump spillways, power stations curved in plan, and dams of spherical form.

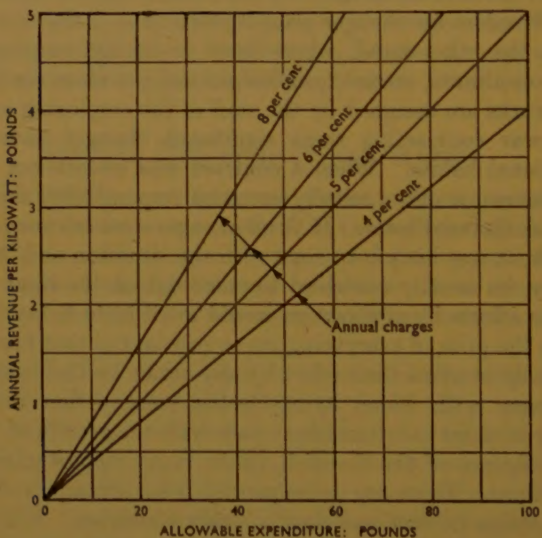
Form of Developments

Water-power development in Scotland has been either for the special purpose of producing aluminium or for public supply. For such a continuous process as aluminium production the development must aim at producing as nearly as possible the same output each day of the year. For

public supply, more output is required during the winter than in the summer, during weekdays than at weekends, and during working hours than at night. This kind of demand is normally met by having generating plant with a capacity of about 2½ times what would be needed to produce a uniform output all the year round.

Power produced to meet the needs of public supply has both a kilowatt and a unit value. This is recognized in the Hydro-Electric Development (Scotland) Act, 1943, where the selling price to what was then the Central Electricity Board is divided into a price for kilowatts and a price for units. The unit, or average annual output of any scheme, varies little with the capacity of plant installed, but within limits set by economical and

Fig. 3



Allowable Capital Expenditure per Incremental K.W. for Various Rates of Annual Charges

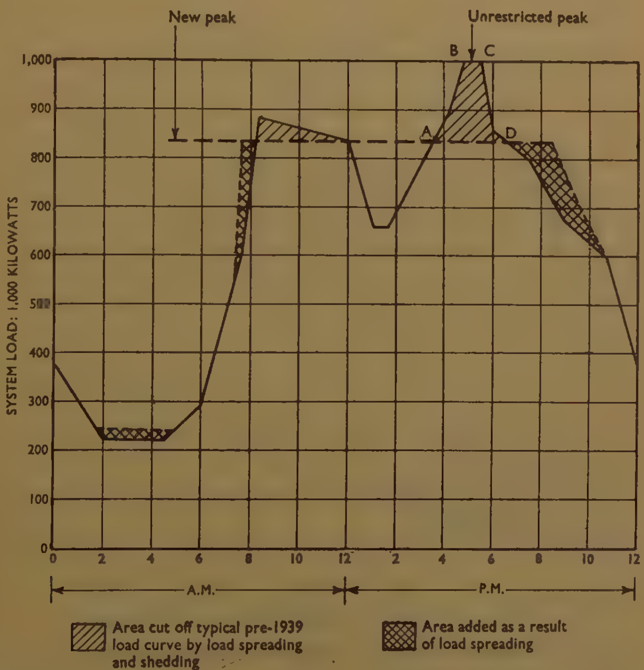
physical conditions, the kilowatts or installed capacity can be varied up or down.

A variation in installed capacity not only alters the size and cost of the generating plant, but also of the aqueducts feeding it. If the extra income produced by the sale of additional kilowatts is greater than the extra charges for larger plant and aqueducts, then there is at least an economic justification for increasing the installed capacity. *Fig. 3* shows how much can be spent on additional kilowatts at different kilowatt prices and rates of annual charges.

The economic, however, is not the only consideration which decides

the capacity of any station. For one thing the load-demand of a system requires the minimum number of units in association with its maximum kilowatt demand. Therefore kilowatts without a sufficiency of units in support have little value. Today the proportion of kilowatts to units is less than it was, owing to the change which has taken place in the daily-load curves as a result of exhortations to keep off the peak and by load shedding and spreading.

Fig. 4



Change in Daily Load Curve effected by Load Shedding and Spreading

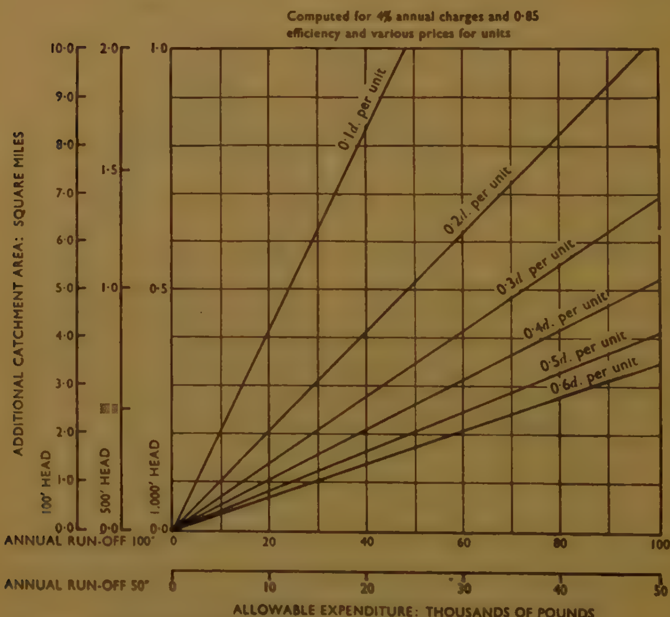
Fig. 4 shows approximately the change which has occurred in the daily load curve and illustrates how in present circumstances by the elimination of area ABCD there is less need for extremely low load factor stations.

Value of Diverted Water

The intensive diversion of water, which is a characteristic of Scottish schemes, is mostly practised where there is both high intensity of rainfall and high head. A decision to bring in extra water depends on whether

the extra revenue produced by such water is sufficient to pay the annual charges on the extra costs involved. *Fig. 5* shows, for different prices of electrical units, the extra capital cost which could be spent to bring in additional catchment areas for different run-offs and heads. For a head of 1,000 feet and a catchment having a run-off of 100 inches per annum it would be profitable at a price of 0·5*d.* per unit to bring in an extra 0·4 square mile, provided that no more than £97,500 was spent in doing so. Similarly for a catchment whose annual run-off was only 50 inches it

Fig. 5



Allowable Capital Expenditure in Additional Aqueducts

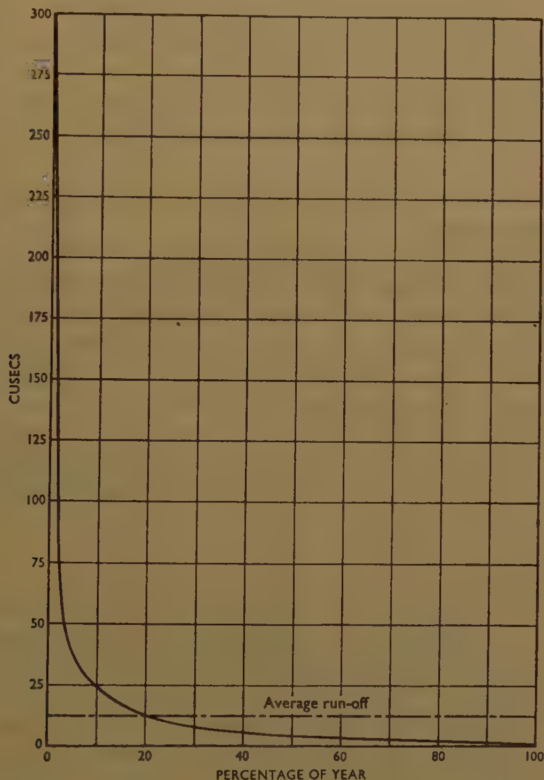
would be necessary to bring in 0·8 square mile, and if the head was only 100 feet the area would need to be 8 square miles.

Size of Diversion Aqueducts

Streams diverted in high-head and high-rainfall areas are necessarily "flashy," with the typical duration curve shown in *Fig. 6*. It is not unusual for the dry-weather flow to fall as low as 0·1 cusec per square mile and the flood flow to rise to about 30 to 40 times the average. For aqueduct capacities in excess of 4½ times the average, little extra water is gained in an average year by increasing them still further, and could only

be justified if the length of aqueduct is a very short one. Fixing the size of a collecting aqueduct in relation to the average flow of the intercepted stream takes no account of wet or unusual years or parts of years. If, instead, the aqueduct size is fixed in relation to the intensity of rainfall which it is designed to catch, the justification would depend on how often the selected intensity is likely to occur. For example, a run-off at a rate

Fig. 6



TYPICAL DURATION CURVE FOR A "FLASHY" STREAM

of 3 inches in 24 hours would, in an area with a rainfall of 90 inches, represent nearly 12 times the average rate of flow and could not be substantiated on inspection of the duration curve (see *Fig. 6*).

IMPOUNDING OF WATER

For reasons already given large scale impounding is not easy in Scotland. When examined on the basis of electrical units stored (which allows for

the elevation of the storage) the present-day cost of impounding water is high, as shown in Table 3.

TABLE 3

Date	Scheme	Electrical units stored *	Capital cost for impounding per unit stored	Annual cost per unit stored
Pre-1939	Ericht	120 millions	0·3*d*.	0·012*d*.
Post-War	Fannich	72 ,,	0·7*d*.	0·028*d*.
,,	Mullardoch	78 ,,	8·3*d*.	0·33*d*.
,,	Sloy	20 ,,	15.0*d*.	0·60*d*.

* Including value through lower stations (if any).

In all cases, however, the cost per unit is less than it would be from thermal stand-by plant.

In consequence of the improved regulation of the flow of streams and rivers by new impounding works, extreme flood conditions in the lower reaches of all main rivers will be eased and a steadier and more uniform flow will obtain throughout the year. Any reduction in flooding is an advantage, but too great an increase in the summer level of a river may have a noticeable effect on agricultural drainage systems which today work at their best at relatively low summer levels. The remedy of lowering the general level of river beds may be already overdue and might have a more lasting effect, owing to the improvement in the flood position.

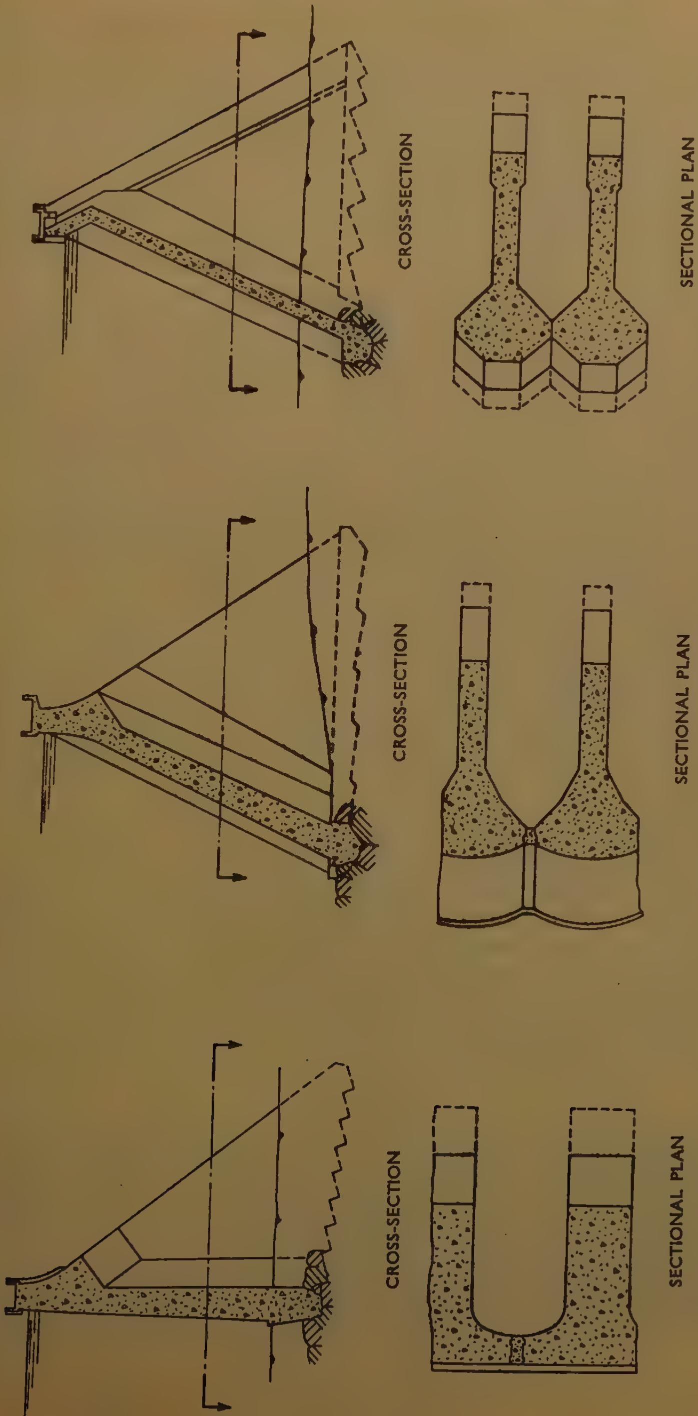
DAMS

In post-war construction the only departures in Scotland from such well tried designs as plain gravity and rock- and earth-fill dams have been several forms of buttress dam. They all possess the same advantage of cutting down the mass of concrete which has to shrink when cooling, and in designs possessing a wide base the intensity of loading is less than with a mass gravity design. *Figs* 7 show the main features of the three designs which are being used, namely :

(*a*) massive buttress at Sloy,
(*b*) round-headed buttress at Shira, and
(*c*) diamond-headed buttress at Errochty.

Gravity dams, which were used at Affric, have made less call on difficult form work and have demonstrated, by the rate of 5,360 cubic yards of concrete placed in a week, that they offer a better opportunity for concentrated concrete-placing than any other design. The substantial extra of about £1 per ton demanded for low-heat cement prevented any substantial employment of this material.

Figs 7



Diamond-Headed Buttress Dam

Round-Headed Buttress Dam

Massive-Buttress Dam

The continuing shortage of cement and the scarcity of carpenters is now directing attention to ways of saving cement, and to rock- and earth-fill dam structures. To economize in cement an experiment is to be made with the use of Fly Ash, as has been done in the United States. Even greater savings in cement are possible if wet-ground blast-furnace slag is used, as pioneered in France. The construction of a dam using this material is contemplated. The slag is delivered in granulated form (controlled by the amount of water used to cool it as it emerges from the furnaces) and ground wet at site. The slurry so obtained is delivered into the mixer and, together with approximately 25 per cent of Portland cement, makes up the normal proportion of cement to aggregate and sand. The French say that the quality of slag they are using reduces heating by a third and not only produces a concrete developing equal strength but one said to be more resistant to aggressive water than when 100 per cent of Portland cement is used. The slag needs no care in storage, is cheap to buy, and can be handled easily in bulk, but must be carefully checked for consistency of chemical content. Only a dam requiring a large quantity of concrete justifies the cost of bringing the special grinding plant on to the site. In Scotland regularity and consistency of supplies of slag would have to be assured before it could be used with confidence.

In abandoning concrete in favour of rock-fill dams, certain decisions are necessary. In one case a rock-fill design has been chosen using an upstream watertight membrane with a small amount of fill behind it, rather than one with a central core-wall of concrete and a much greater amount of fill. In all " fill " structures compaction is vital but the need is even greater with an upstream cut-off. There are several ways of achieving adequate consolidation; the most certain and satisfactory seems to be a form of vibrating table.

It is not always easy to find a satisfactory way of discharging spill water past a rock-fill dam. A gravity concrete spillway section can be interposed between two wing sections of a rock-fill dam, or the spill water can go either via a bellmouth spillway through a tunnel below the dam, or by a long channel spillway at right angles to and at one end of the dam. The dams of this type which are to be used are still at the design stage and a final decision about spillways has not yet been taken.

TUNNELS

The preponderance of schist rocks over a large part of Scotland makes tunnel-driving difficult. Schists vary from soft phyllites to much harder varieties interspersed with bands of hard quartzite. Drilling is relatively easy in the soft varieties but explosives are less effective. As much as 7·8 lb. per cubic yard have been used in some cases compared with 4·5 to 5 lb. in average conditions. On the other hand, drilling has given much trouble in quartzite. Granite is not so troublesome either in drilling or

blasting. In addition, it is useful as an aggregate and as a source for sand, for none of which purposes can schists be used.

Where the rock conditions have been good enough, economies and other advantages have been gained by doing without surface pipelines and supplying the station instead by high-pressure tunnels, which either come out horizontally from a vertical shaft fed at the top by the low-pressure tunnel, or run upwards at an angle to connect to the low-pressure tunnel. In either case they are steel-lined where there is insufficient rock cover to resist the working pressure. Steel lining also prevents loss of water.

After making all allowances for the rise in wages and materials the cost of excavation of tunnels since the war proved higher than had been expected. That was probably due to the scarcity of good tunnellers when the first tunnels were let and to delays in procuring the lighter boring-rigs which are now generally used. Since construction first started, and more particularly since these lighter boring-rigs, with their reduced manning, came into use, there has been a steady improvement in rates of excavation. From the rather poor average progress of 10 cubic yards per man per week for tunnels within the 10–15-feet-diameter range, average speeds are now being attained which give 30 cubic yards per man per week. If wages and prices had been more stable it should have been possible to see a decisive lowering of costs as a result of greater skill, the use of fewer men, and the application of better mucking equipment of greater capacity. Since an average mucking occupies two-thirds of the time taken for each cycle of operations, it affords the best opportunity for cutting down the cycle time, whereas improvements in drilling are most effective in reducing the manpower needed.

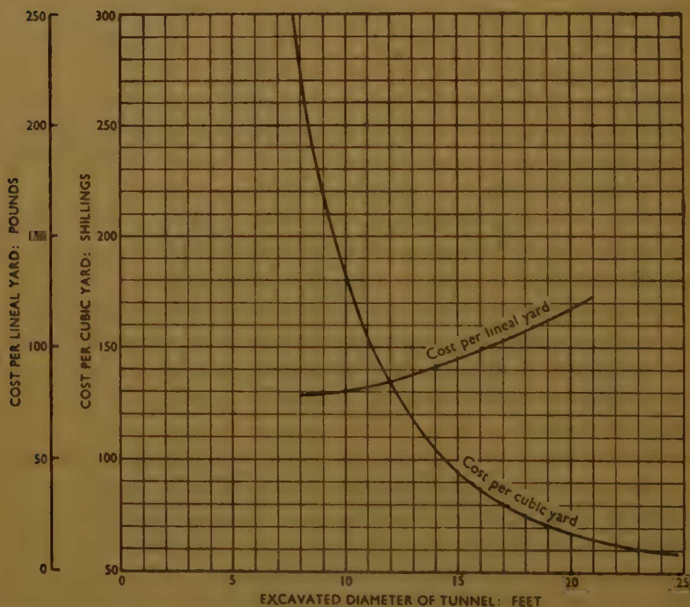
The working space required for the smallest mechanical loaders fixes the minimum size of tunnel which can be driven with their help. With either hand or mechanical loading the cost of excavating a cubic yard of rock goes up rapidly with every reduction in size, and, as will be seen from *Fig. 8*, the cost per lineal yard shows no reduction below a diameter of 8 feet and smaller diameters are not economic.

At the present level of costs it is still more attractive economically to line a tunnel with concrete than to use a larger unlined tunnel. For equal carrying capacity the diameter of an unlined tunnel requires to be 1·40 times that of a lined tunnel. So long as the cost of the concrete lining is cheaper than the cost of excavating 96 per cent more rock, then a lined tunnel is justified on economic grounds. Any tunnel delivering water direct to the water turbine plant is better to be lined, but a tunnel collecting water from a diversion, transferring it from one reservoir to another, or being used as a tailrace need not necessarily be lined. An unlined tunnel of a larger size, if subsequently lined, would automatically gain in carrying capacity. Most contractors are now placing their tunnel-lining concrete by pump or by pneumatic placers.

Aqueducts

Where pressure tunnels have not been possible, either because of poor rock conditions or for reasons of size, steel pipelines have been used to convey pressure water to the stations. Owing to its scarcity some alternative to steel for pipelines would have been useful. Pre-stressed concrete, as is being used in France for a pipeline 17 feet in diameter and for a pressure of 330 feet, might have been the answer had not both cement and reinforcing steel been in short supply also. That development is, however,

Fig. 8



Relation between Unit Costs of Excavation and Diameter of Tunnel

being watched. In Sweden, engineers have chosen to put many of their power stations almost vertically under the dam, and thereby have been able to reduce the length of pressure aqueducts to the absolute minimum. Such a solution introduces difficult problems of access and so far these have prevented the adoption of the idea in Scotland.

Low-level surface aqueducts to carry large quantities of water are normally of open trapezoidal form, lined to reduce leakage and improve the discharge coefficient. In winter, at high elevations, snow and ice are liable to choke open canals and so spun-concrete pipes are being used instead. There is a considerable reduction in on-site concreting when

pipes are used, but when the pipes are large the transport problems to high and remote sites can be considerable.

Collecting tunnels and aqueducts may require to have spill points if the intakes at the streams they are intercepting do not provide for the discard of flows in excess of the aqueduct's capacity. It is probably advantageous to provide for spill in that way because it allows for some local diversity in rainfall and run-off.

Intakes and Screens

The design and size of a main tunnel intake are affected by the range of draw-down to be used and by the screening requirements. If the screens have only to exclude the size of trash which cannot pass through the turbine runner the spacing is usually wide (especially with Kaplan turbines) and the intake dimensions are scarcely affected. If, however, descending fish—smolts in particular—have to be excluded, the spacing between bars must be less than $1\frac{5}{8}$ inch for salmon and $\frac{1}{2}$ inch vertical by 1 inch* horizontal for smolts. Also if smolts and kelts are not to be pinned against the screens and killed the limiting velocity through the screens at lowest draw-off level should be not more than 1 foot per second. To comply with these limitations the screen area required to stop smolts is very great. Not only are the screens large and cumbersome and difficult to clean, but the dimensions of the intake go up proportionately. Where screens are needed at the mouth of the tailrace to keep ascending fish out of the station, the spacing used is generally $1\frac{5}{8}$ inch and presents less of a problem. Where there is no tailrace and the suction tubes discharge direct into a river course or a loch, the turbulence of the water may suffice without screens to deter fish. Experiments which are proceeding with electric fish screens indicate that ascending fish can be kept out of certain waters. The results are not so satisfactory for descending fish ; there is a risk that a fish which comes close enough to get a shock may be unable to make its way upstream again out of the electrified area.

Control Gates and Valves

Control gates and valves are being standardized as far as possible. Main control gates are of the fixed or free-roller type operated by motor. The power for closing, if not provided by the motor, is obtained instead by the out-of-balance between the gate and its counter weight. When closed by gravity, suitable braking either by oil-pump retarders or solenoid brakes is provided. Protection for the main gate is by an upstream emergency gate of the tube-rolling type, closing by gravity under brake control. Float-controlled hydraulically operated drum gates have been employed successfully as flood-control gates. Little use has so far been made of the sector design of gate which has proved popular in other

* $\frac{3}{4}$ inch in original MS. Later corrected to 1 inch.

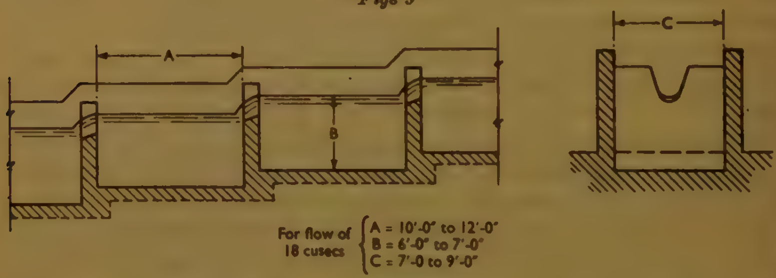
countries for use on overflow crests and for all sorts of canal and intake control.

Where pressure pipelines are used they are controlled at the tunnel portal by butterfly valves fitted with over-velocity and remote trip controls. With pressure tunnels, on the other hand, there is no portal so there are no control valves between the turbine inlet valve and the main tunnel intake gate. This means that nothing can be done on the pressure part of the system near the power station without emptying the tunnel.

Fish Passes

The provision for the ascent of fish past power stations and dams has, until recently, been by fish-ladder—either of the overfall or submerged-orifice type. The overfall pass is of well-established and straightforward

Figs 9



Fish Pass—Overfall Type

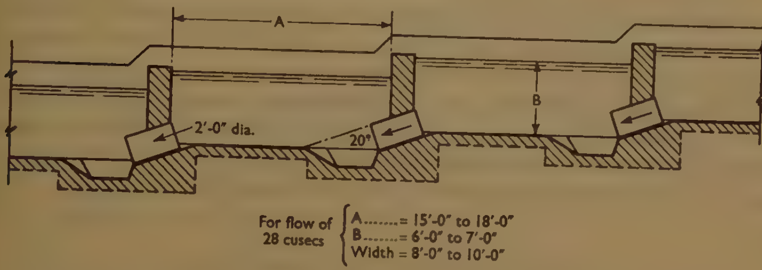
design. A limiting difference in level between pools of 18 inches was usual and, provided that the minimum dimensions of the pools were as given in *Figs 9*, fish had no apparent difficulty in making the ascent. For large flows and for important rivers the submerged orifice design as reviewed by the Institution Fish Pass Committee has been developed and used. Here again, a head difference between pools of 18 inches has been generally adopted, and the flow through the orifices is directed into a spoon-shaped depression in the floor. The governing dimensions for the pools to keep turbulence to the minimum are given in *Fig. 10*. They were obtained by experiment at the City & Guilds College, London, by Professor C. M. White and checked in a full-scale model before any passes were built. Several are now in successful use, as proved by a count of the fish using them. Through windows in the side of a pool the fish were counted as they passed from one pool to the next. In 1951, 5,600 salmon were counted ascending the Pitlochry Pass.

A new type of pass has been developed by Mr. Borland of Kilmarnock, and tried out at Leixlip on the River Liffey in Ireland.[1] In principle it

[1] "The Hydraulic Fish Lift at Leixlip." I.C.E.I. Bulletin, Apr. 1951.

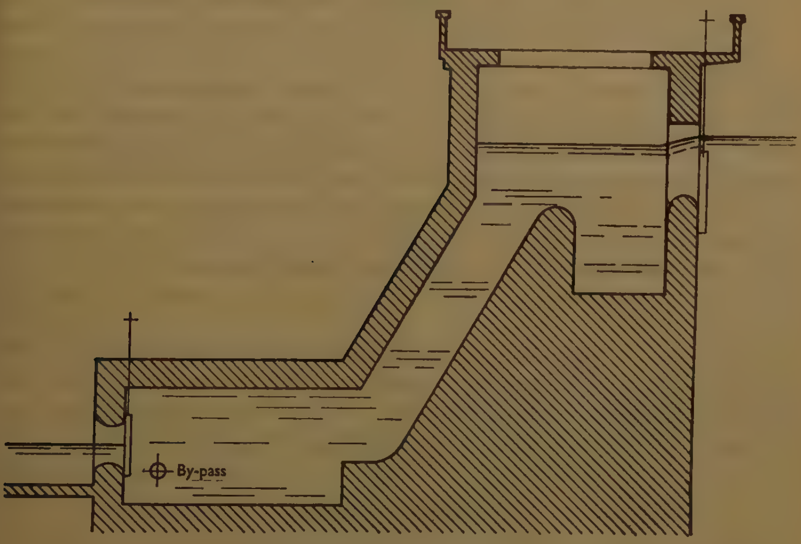
is different from the pool types, for it calls for no effort from the fish using it. They are raised from the tail water to the upper water level in one operation by flooding the chamber shown in *Fig. 11*. This type of pass is not very costly, requires little water, is readily incorporated as part

Fig. 10



FISH PASS—ORIFICE TYPE

Fig. 11



FISH PASS—BORLAND TYPE

of a dam structure, and has proved very successful in Ireland. Between the beginning of May and the middle of October 1950, 2,375 fish were counted ascending. Borland passes are to be used on several Scottish schemes now under construction.

Surge Shafts

Surge shafts or tanks are needed in a hydro scheme to give time for water to accelerate or decelerate as the case may be when load changes take place. They are usually large and expensive structures. The interlinking of more and more stations has greatly reduced the chance of a partly loaded station connected to the grid getting a sudden substantial addition to the load it is already carrying. Also a set or a station which has discarded its load by tripping off from an interconnected system cannot be reloaded without first re-synchronizing, and so affords an opportunity to rebuild the load under careful control. In these circumstances there is an opportunity to adjust the design of surge tanks, should physical conditions limit the dimensions for such structures.

Concrete and Building Materials

Most Scottish rivers draw their water from peaty uplands and so are usually on the acid side of neutrality. Since such water is aggressive to lime and lime cements, precautions are necessary to limit its effect on the concrete water-retaining structures. Despite the enthusiasm with which it was introduced, experience with alumina cement has not provided a satisfactory answer to this problem. Instead a rich concrete, with a mix by volume of one of cement to four of dry graded aggregates, is preferred for surfaces in contact with water. In addition, bitumastic paint is generally applied to exposed surfaces. Because of the risk of deterioration, not only of the concrete but also ultimately of the reinforcement, reinforced concrete has not been considered for such important water-retaining structures as dams.

The concrete mix (by volume) generally used for mass concrete not less than 2 feet away from a water surface is one of cement to seven of aggregates.

Power-station buildings are being faced either with good quality pre-cast slabbing or natural stone. Solid walling of natural stone is to be used increasingly for future stations, with the object of restoring some increase of prosperity to the stone quarrying and mason industries.

Generating Plant

The civil engineer's interest in the generating plant is not merely to provide the supply needed by the machines ; he is also interested to see that those installed will, if correctly coordinated, produce the maximum number of kilowatt-hours from the cusecs he makes available. The flexibility of operation now afforded by an increasing number of interconnected stations makes multiple-set stations less necessary and, in addition, the development of the Kaplan with its highly sustained efficiency over a wide range

of loads has made single-unit stations, with the resulting simplification in electrical control, very attractive.

Selection of Schemes

The nature of the load to be served, a scheme's convenience to that load, and the relation of that scheme's probable cost of production compared with steam-produced power are the main considerations in selecting schemes for promotion and construction. It has generally been the case that when a particular hydro scheme has proceeded in Scotland it has appeared to be only a little more economic than steam generation. With the passage of time and the change in the value of money all schemes have proved progressively more economic. Schemes which have proceeded can be put in the following categories :—

High-Head Schemes

High-head schemes, such as Sloy, drain small but very wet areas and are attractive, provided the main tunnel is not long and the size of the dam required is reasonable. They depend largely on diverted water and are usually suited to low load factor working.

River-Basin Schemes

River-basin schemes, such as Tummel-Garry and the River Conon, involve a whole watershed and usually embrace a selection of developments utilizing heads from 600–700 feet down to such low heads as 25 feet. Their value lies in a stage-by-stage utilization of the storage created in the upper tributaries or headwaters. In such groups of stations it is usual to apportion the cost of storage over all the stations through which the regulated water passes, instead of debiting it all to the section to which it belongs.

Isolated Schemes

Isolated schemes, such as Morar and Lochalsh, are small developments for local supply. They are not supported by any connexion to the grid, so require to be as nearly self-contained as possible, with either adequate storage or stand-by diesel plant to tide over dry spells.

Assessment of Output

All assessments of output from potential schemes are on the basis of the kilowatt-hour output in an average year, and, in the almost complete absence of long-term flow records, these estimates rely on the long-term isohyetal curves prepared by the Meteorological Office. Excluding the schemes which had been constructed prior to 1943, the potential power in the Highlands was estimated in that year as being more than 6,000 million

kilowatt-hours. It may be that this figure will be substantially exceeded if the demand for electricity continues to grow at its present rate and if coal prices go high enough. Also, most schemes as they are developed are producing more output than was forecast in the estimates.

Stage reached in Development

In the period between the beginning of 1944 and the end of 1951 the progress made with the survey, promotion, construction, and completion of schemes is as shown in Table 4.

The total of 2,849·5 millions of kilowatt hours is nearly half what was estimated to be the potential output in 1943, and approximates to the consumption of the whole of Scotland at that time. Since then, however, the consumption of electricity has grown and is now about 70 per cent greater.

It is not possible to refer, except in the following brief way, to some of the schemes listed in Table 4, whose locations are given in *Fig. 12*. Possibly the various engineers concerned with the detailed designs of these schemes will contribute, from time to time, detailed descriptions of the works and methods of construction to meetings of the Institution or its Divisions.

Sloy

The Sloy scheme is a good example of a high-head high-rainfall scheme with extensive collecting aqueducts. The natural catchment of Loch Sloy, which extends to only 6·5 square miles, has been increased to 32 square miles by a series of collecting aqueducts, having a total length of 20·5 miles. This gives a total of 0·8 mile for every square mile of additional catchment brought in. *Fig. 13* (facing p. 272) is a view of the dam, which is of the massive buttress type, designed by Mr James Williamson, the Engineer for the Works. Two 10-ton overhead cableways, capable of traversing almost the whole plan area of the dam, were used to place concrete. Lifts were made 5 feet high but were placed at an upward angle to the horizontal to prevent shear on the construction joints. Arch rings, spillway decking, and other items such as parapets, were pre-cast on the ground and lifted by the cableway into position in order to avoid difficult in-situ shuttering. Practically all rock excavated for the work was schist and unsuitable as aggregate for concrete. A special quarry was opened in an epidiorite outcrop about 1½ mile away, and the crushed stone was conveyed by belt conveyor direct to the dam.

Apart from two faults which met on the line of the tunnel and made it necessary to deviate the tunnel line to get sound rock, the tunnel offered no serious problems. The amount of water encountered was not excessive. At 26 feet, the surge-shaft diameter was the maximum considered safe in the rock encountered. A larger diameter would have been used otherwise.

The four pipelines are stepped down from 7 feet internal diameter at the top to 6 feet $3\frac{7}{8}$ inches at the power station, the thickness increasing from $\frac{11}{16}$ inches to $1\frac{9}{16}$ inches. They are in 24-foot lengths, each made up of three 8-foot-long electrically shop-welded strakes and having site-riveted

Fig. 12

THE SCOTTISH HYDRO-ELECTRIC SCHEMES

strap joints between each length. At saddles, the pipes are carried on bearing strips of phosphor bronze.

The dimensions of the dam and tunnel and particulars of the quantities

TABLE 4.—HYDRO-ELECTRIC SCHEMES

	Capacity: kilowatts	Annual output: kilowatt-hours
In operation		
Sloy	130,450	120,000,000
Nostie Bridge (Lochalsh)	1,250	4,000,000
Morar	750	2,000,000
Pitlochry } Tummel-Garry	15,000	54,000,000
Clunie } Tummel-Garry	61,200	143,000,000
Grudie Bridge (Conon Basin)	24,000	83,000,000
Affric *	44,000	150,000,000
Cowal	6,000	14,000,000
Total (i)	282,650	570,000,000
Under construction		
Errochty (Tummel-Garry)	75,000	103,000,000
Gairloch	750	3,000,000
Affric *	24,000	80,000,000
Shira	45,000	80,000,000
Lussa (Mull of Kintyre)	2,000	8,500,000
Storr Lochs (Skye)	1,600	3,500,000
Gaur (Tummel)	6,000	17,000,000
Glascarnoch } Conon Basin	24,000	112,000,000
Luichart } Conon Basin	24,000	124,000,000
Torr Achilty } Conon Basin	15,000	36,000,000
Achanalt } Conon Basin	2,000	8,000,000
Quoich } Inverness Garry group	20,000	77,000,000
Invergarry } Inverness Garry group	20,000	82,000,000
Total (ii)	259,350	734,000,000
Promoted and just starting construction		
Ceannacroc } Moriston group	17,500	70,000,000
Doe } Moriston group	2,700	7,000,000
Invermoriston } Moriston group	24,000	105,000,000
Livishie } Moriston group	10,900	21,000,000
Lawers	30,000	80,000,000
Total (iii)	85,100	283,000,000
Promoted		
Allt-na-Lairige (Shira)	6,000	17,000,000
Loch Dubh (Ullapool)	600	1,500,000
Total (iv)	6,600	18,500,000
Under promotion		
Mucomir	1,500	9,000,000
Orrin (Conon Basin)	18,000	79,000,000
Kilmelfort	3,000	9,000,000
Shin (Sutherland)	44,000	168,000,000
Breadalbane (Perthshire)	88,500	304,000,000
Total (v)	155,000	569,000,000
Under survey		
Farrar, Beauly, Tay Basin, Affric extensions, Awe.	175,000	675,000,000
Total (vi)	175,000	675,000,000
Grand total (i) to (vi)	963,700	2,849,500,000

* The Affric Scheme, being partly completed, appears under two different headings.

of rock excavated and concrete placed are given in Tables 5 (*a*) and (*b*), together with the corresponding figures for works in other schemes.

Table 5 (*a*).—DETAILS OF MAJOR DAMS COMPLETED OR ON WHICH CONSTRUCTION IS WELL ADVANCED

Scheme	Dam	Type	Maximum height above river bed: feet	Length: feet	Concrete: cu. yds	Water impounded: cu. ft. $\times 10^6$
Sloy . .	Sloy	Concrete massive buttress	165	1,170	208,000	1,240
Tummel-Garry	Pitlochry	Concrete gravity	54	475	67,000	415
	Clunie	Concrete gravity	65	386	41,000	910
	Errochty	Concrete diamond-headed buttress	130	1,300	220,000	1,150
Affric .	Mullardoch	Concrete gravity	116	2,385	275,000	7,470
	Benevean	Concrete gravity	86	516	65,000	970
Shira .	Upper	Concrete round-headed buttress	133	2,250	260,000*	800

Table 5 (*b*).—DETAILS OF MAIN TUNNELS DRIVEN

Scheme	Tunnel	Length: feet	Equivalent finished diameter: feet	Cross-section	Lined or unlined
Sloy . .	Sloy	9,970	15·8	Mainly horse-shoe	Lined
Tummel-Garry	Clunie	9,200	23	Mainly horse-shoe	Lined
	Errochty	31,700	15·5	Mainly horse-shoe	Lined
Fannich .	Fannich	19,400	10	Mainly horse-shoe	Lined
Affric. .	Mullardoch	17,200	15·75	Horse-shoe	Partially lined
	Fasnakyle	17,330	14·5	Horse-shoe and circular	Lined
Shira . .	Shira	23,400	10	Circular	Lined

Tummel-Garry

Apart from a new impounding dam on the River Errochty, the Tummel-Garry scheme utilizes water already regulated by the earlier Grampian

* Corrected from 300,000; see pp. 293 and 301.

Works.[1] The impounding dam on the Errochty is of the diamond-headed buttress type. A feature of this type of dam is its low intensity of loading. It was fortunate that this type was selected beforehand, because as excavation proceeded the foundation rock was found to have a much lower bearing value than expected, and to be interspersed with bands of clay. The tunnel from the dam to the power station on the shores of Loch Tummel is 6 miles long. On the way, it passes under a stream below which there is much overlying drift to rock. The tunnel line was lowered sufficiently to come wholly within rock. Since there is no pipeline and the tunnel continues to the power station in a high-pressure section, the disadvantage of having to go so low at this stream was not serious. The shaft from the main tunnel to the high-pressure tunnel, and its continuation to the bottom of the surge shaft is 486 feet deep and 15 feet diameter. In excavating it, two helical pilot-shafts were driven from the bottom at an angle of 180 degrees to each other. Feeding the reservoir there is a total of 11·5 miles of collecting tunnels, with sizes ranging from 8 feet 6 inches equivalent diameter, partially lined, to 10 feet equivalent diameter, fully lined.

The Clunie section of the Tummel-Garry Works produces at its power station 143 million kilowatt hours under the moderate head of 173 feet. The main tunnel has a finished equivalent diameter of 23 feet and a maximum capacity of 5,100 cusecs. It is probably the largest water tunnel in Britain. *Fig. 14* shows drilling in progress. The surge shaft, with a diameter of 110 feet, is also one of the largest in Great Britain. The intake to the Clunie tunnel has been designed with screens to exclude smolts and consequently is a structure of considerable size.

The Pitlochry station is illustrated in *Fig. 15*. It is the lowest station in the group, and was designed to deliver as uniform a discharge down the River Tummel as possible. It is equipped with Kaplan turbines to enable the station to run efficiently over a wide range of discharges. Whilst the dam proper is not a large structure, a substantial concrete cut-off wall was needed to render watertight a raised beach of sand and gravel on the east bank of the river. The length of this cut-off was greater than that of the dam itself and the excavations had to go down deeper to reach rock because the gravel bank concealed an old river channel, at a lower level than the present one. An approximate profile of the rock at the site is shown in *Fig. 16*.

Fish ladders of the submerged orifice type are provided at Pitlochry and Clunie Dams for the passage of fish. That at Pitlochry is the larger, with a capacity of 65 cusecs compared with 45 cusecs for Clunie. In both cases the compensation flows which must be released are greater than in the fish passes. The differences are passed through compensation-water

[1] A. S. Valentine and E. M. Bergstrom, "Hydro-Electric Development in Great Britain, with Special Reference to the Works of the Grampian Electricity Supply Co." J. Instn Elec. Engrs, vol. 76, p. 125 (Feb. 1935).

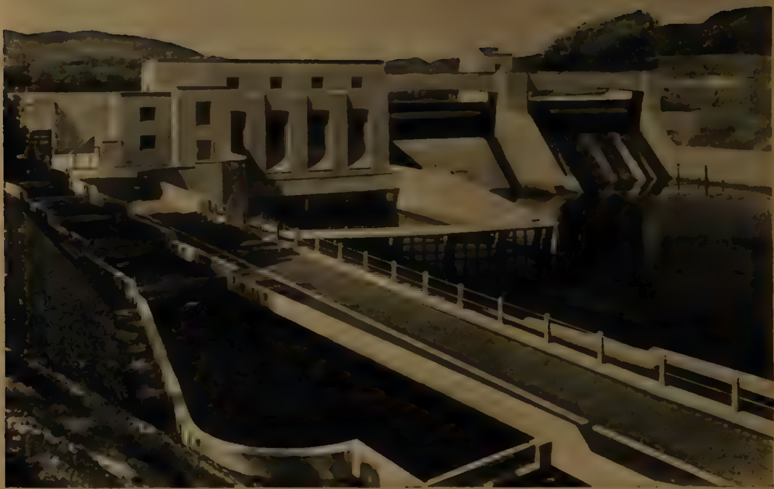
Fig. 13

SLOY DAM

Fig. 14

TRAVELLING GANTRY, MOUNTING FIFTEEN I.R.505 DRIFTERS MOVING TO FACE.
CLUNIE TUNNEL, 5 AUG. 1948

Fig. 15



PITLOCHRY DAM AND POWER STATION

Fig. 18

MULLARDOCH DAM. VIEW FROM GATESHAFT AREA LOOKING NORTH.
7 NOV. 1950

turbines which discharge into the lowest pool of the fish passes to give the maximum attraction for ascending fish. The fish pass at Pitlochry is seen in the foreground of *Fig. 15.*

Conon Basin

The Fannich Scheme (now in operation) represents the first stage of the comprehensive development of the Conon River. It is a simple scheme, for no dam is provided at the outlet of Loch Fannich. This is a deep loch and storage of 60 per cent of the average annual output is obtained by drawing down the level by 50 feet. The construction of the tunnel and other works was quite straightforward, but the blowing-through of the underwater intake to the tunnel was an undertaking of some difficulty. However, by employing the same contractors, advantage was taken of the experience gained at the similar operation in Loch Treig. In this case it was necessary to have a large sump for the reception of the blown material. The general arrangement adopted is shown in *Fig. 17.* It also shows the two shafts which were used. The one nearer the loch is the main gate shaft. The second one, now the screen shaft, was used for access to the temporary concrete plug in the tunnel between the two shafts. This plug, and not the main gate, was used to hold back the water after the break-through. The whole operation was very successful and reflected great credit on the Engineers, Sir Alexander Gibb & Partners, and the Contractors, Balfour, Beatty & Co., Ltd.

The Conon is a salmon river, so that provision for the ascent of fish is required at four of six main dams. Torr Achilty and Luichart, the lowest stations of the group, are similar in many respects to the Pitlochry and Clunie Station in the Tummel-Garry group. The two turbines at Torr Achilty are to be duplicates of the two Kaplans at Pitlochry.

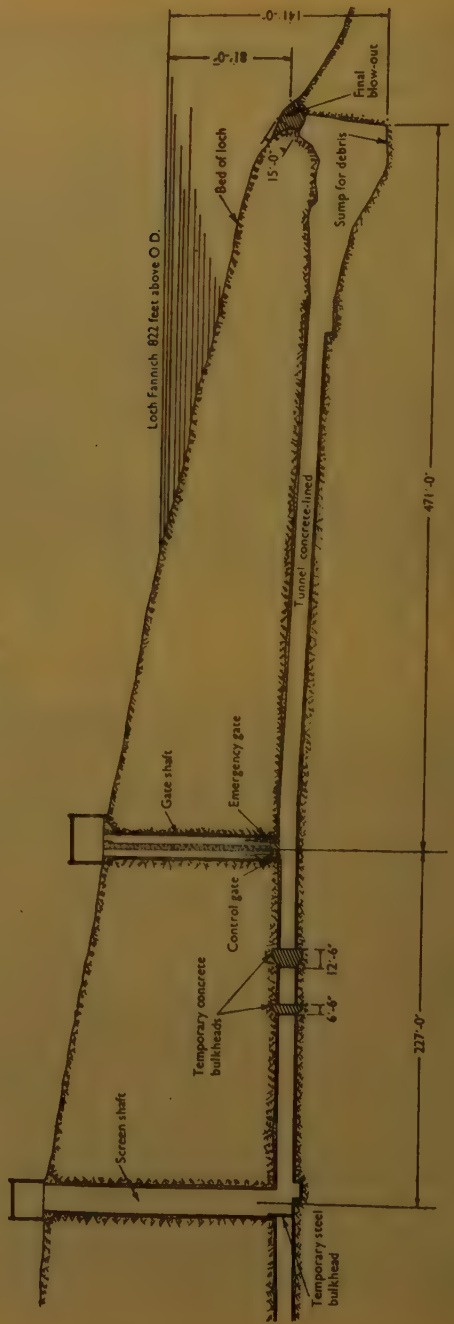
Affric Scheme

The Affric scheme develops the output from the two parallel valleys of Glen Cannich and Glen Affric. The main storage is provided by a dam 116 feet high above the river bed at the outlet of Loch Mullardoch in Glen Cannich. This dam is of gravity section, slightly V-shaped in plan with the apex downstream on an island about midway across the outlet of the loch. At one stage, in order to make savings, it was decided to construct part of the dam to 20 feet short of its full height. A cross-section suited to this reduced height was used for part of the dam, but ultimately it was decided it would be more economical, while the contractor's plant was still on the site, to go to the full height over the whole length of the dam. This is being done by applying, wherever the smaller section had been used, a concrete blanket 11 feet 6 inches thick to the downstream face of the dam. This blanket does not rest directly on the dam but on prepacked fill which

Fig. 16

PITLOCHRY DAM AND CUT-OFF WALL

Fig. 17



FANNICH-TUNNEL INTAKE

will be grouted up solid so soon as the old and the new concrete have reached a stable temperature. A view of the dam during construction is given in *Fig. 18*.

The last section of the tunnel supplying the main power station is under full pressure from the dam. A profile of the whole tunnel is given in *Fig. 19* and shows the extent of steel lining which was considered necessary.

Shira Scheme

The Shira scheme is a two-stage development of a total fall of 1,098 feet. A head of 138 feet is utilized in the upper stage and 960 feet in the lower stage. The power station capacities are 5,000 kilowatts and 40,000 kilowatts respectively. The reservoir created by the upper-stage dam provides the main storage, but in wet weather it is expected that sufficient water will reach the much smaller lower reservoir from collecting aqueducts to feed the main power station without drawing any water from the main reservoir. During wet week-ends when the demand for energy is small, output from the lower station may not be needed. In that case, water from the lower reservoir can be pumped back to the upper reservoir by a pump in the upper station. The pump is mounted on the same shaft as the turbine and alternator and will be driven by the alternator running as a motor. Experience with this pump should provide useful data for other pumping projects.

A round-headed-buttress design was selected for the main or upper dam at Shira, because, of the several designs considered, this appeared to be the most economical for the location and site. Although excavation for the foundations is well advanced, no concreting has yet been done.

In this case the high-pressure tunnel to the power station (which will be underground) is a sloping shaft. It was driven almost entirely from the bottom because the slope brought most of the spoil down without much mechanical assistance.

Table 4 shows that a large volume of work is at present under construction. Apart from those schemes dealt with, however, none has proceeded far enough to justify special comment. It is still too soon to say whether the decisions which have been taken, and may be taken, to try out new ideas for overcoming shortages are to be successful. Abnormal conditions have characterized the years since 1944 and have made it most difficult to compare alternative designs and methods of construction on any sort of equality. A proposal turned down because of an apparently adverse difference of 5 per cent in cost might in the end have proved cheaper. Reservations of some sort are inevitable, but it is hoped that after all due allowances have been made the results of any experiments will be satisfactory.

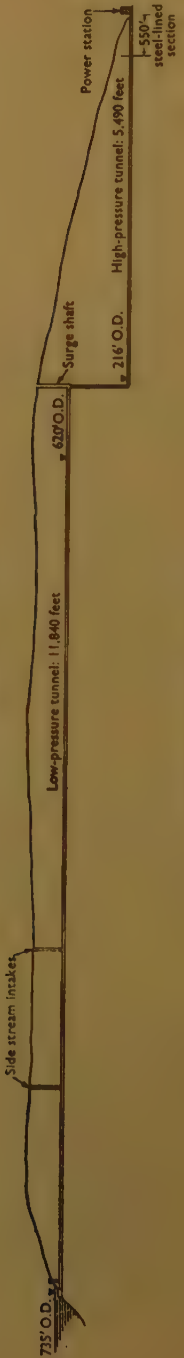


Fig. 19

AFFRIC-FASNAKYLE TUNNEL

ACKNOWLEDGEMENTS

The Author wishes to thank his staff for all their help in the preparation of the Paper.

The Paper is accompanied by four photographs and eleven sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

Mr S. B. Donkin opening the discussion, asked permission of the President and the Meeting to pay a tribute to the late Sir Edward MacColl, who until the previous June, when he had died of overwork and heart failure on the eve of the opening ceremony of the Pitlochry scheme, and had been the Author's chief.

Sir Edward MacColl had been a great engineer, with ability, strength of character, and the strength of his convictions, so that he had been able to satisfy his ambition of benefitting the people of the Scottish Highlands by Scottish inspiration and performance. He disliked any domination or control from London; he was a true Scot.

He had been largely responsible for the selection of the eighty-four hydro-electric projects referred to in the Hilleary Report of 1938, and for preparing information for the Cooper Report in 1942. He had collaborated to a very large extent with Mr James Williamson in the latter Report. All the work of preparation had been done by Sir Edward MacColl in his spare time, while working for the Central Electricity Board in Scotland, and he had "worked himself out"—with great benefit to that country.

He had been able to convince his Board, after being selected as the Deputy Chairman and Chief Technical Officer of the North of Scotland Board, that to begin with they must carry out two or three large schemes as near as possible to the industrial area of Scotland, so as to supply cheap energy to that area as soon as possible and bring in a revenue before less profitable schemes were put in hand—a point of economics for which he was famous. He had received much gratification on account of Her Majesty the Queen having attended and performed the opening ceremony of the Loch Sloy Power Station on 18th October, 1950. The official opening of the Pitlochry scheme had been postponed, but he would have been proud that Lady MacColl was to perform that ceremony on 16th June, 1951.

Those who formed the Advisory Panel had realized the value of his critical faculties when schemes were being investigated, and the members of the Panel always admired his sound knowledge of civil engineering, of

mechanical engineering, and of hydraulic and electrical engineering. It was wonderful that a man should have all those faculties in one body. The success of the projects was largely the result of his ideas and of his great appreciation of the economics of each project.

Finally he was, as many members knew, an inventor. He invented the well-known system for the automatic protection of A.C. systems, and it must be said of him that he was responsible for advising the Board to start the development, by John Brown and Company Limited, of the gas turbine with special reference to the use of peat and peat gas. Up to the time of his decease he had been working on the possible application of the hydraulic air compressor to enable, on appropriate sites, hydraulic power to be used to provide about 60 per cent of the potential energy of the gas turbine with no expenditure of fuel and at a higher efficiency. His mind had never been at rest, and he deserved to go down to posterity as one of the great power engineers of his time.

Mr J. Guthrie Brown pointed out that, under the heading "Control Gates and Valves," the Author had stated : " Float-controlled hydraulically-operated drum-gates have been employed successfully as flood-control gates." That sentence dismissed very briefly the many problems involved in installing two 90-foot-by-16-foot-deep drum gates at the Pitlochry dam, and two 60-foot-by-16-foot drum gates at the Clunie dam on the Tummel-Garry project. The two drum gates at the Pitlochry dam could be seen in *Fig. 15.* They were actually the first examples of that type of gate to be installed in Great Britain.

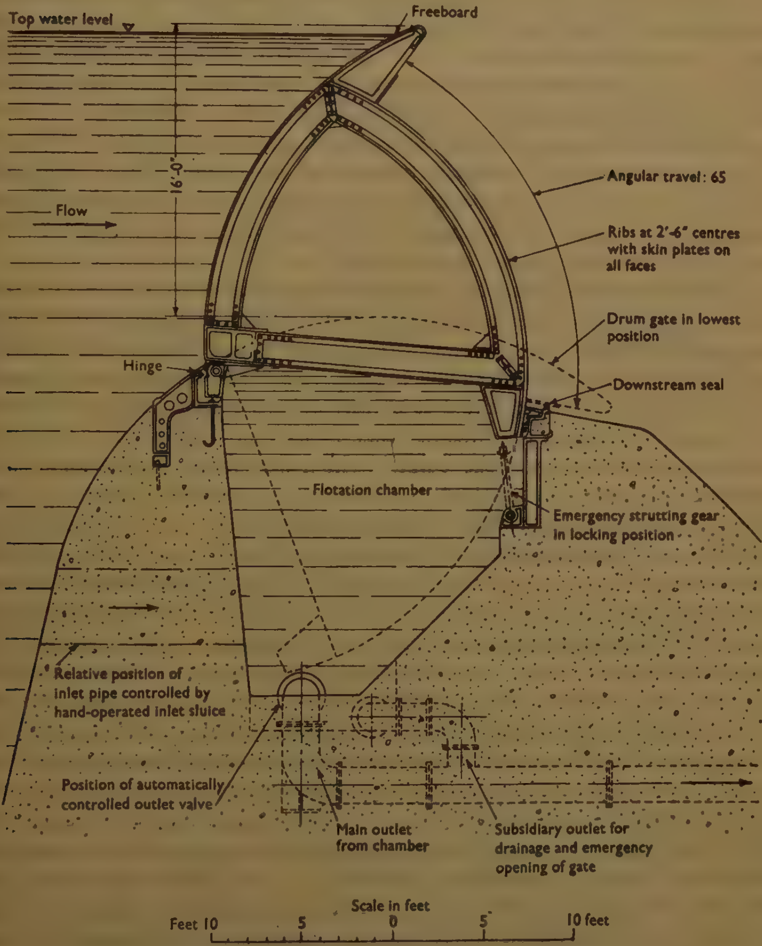
Apart from the question of amenity—a very important matter at Pitlochry—the use of drum gates had been dictated by the need to be able to discharge automatically up to 40,000 cusecs with an insignificant rise in the water-level in the Pitlochry reservoir. With an ordinary overfall weir, allowing a 4-foot rise in the reservoir, it would require a spillway with a length of about 1,500 feet, and that was obviously impracticable. Flood gates of conventional design would have been very large and would have had unsightly gearing and gantries. The drum gates had none of those objections and could easily control a major flood with a rise in the reservoir level of only 12 inches.

Fig. 20 showed the type of drum gate installed at Pitlochry and Clunie dams. The principle was quite simple. The gate was a triangular structure, hinged at the upstream end and kept in equilibrium by means of a flotation chamber. The water pressure on the upstream side of the gate was balanced by the slightly greater water pressure in the flotation chamber ; the resultant outward reaction at the hinge, in the case of the 90-foot-long Pitlochry gate, was about 825 tons. The main criticism which could be made of the gate was that it did involve the use of a considerable quantity of reinforcing concrete near the hinge, which was permanently immersed in water. Very great care had been taken of the concrete construction, and large-diameter post-stressed rods had been put in at intervals to ensure

a greater safety factor. Also, the concrete exposed to the water had been given a very heavy coating of bituminous paint.

The operation of the gate was quite simple. The flotation chamber was permanently connected with the reservoir by means of an opening which

Fig. 20



Tummel-Garry Scheme. Typical Section of Drum Gate

could be adjusted to control the amount of water entering. The flotation chamber was also connected to a 24-inch pipe which discharged downstream. When the water level in the reservoir rose a few inches during floods a float-controlled valve opened, and the water from the flotation

chamber was discharged. That reduced the pressure in the flotation chamber and the gate gradually lowered until equilibrium was established. If the water level filled, the valve re-closed and the gate rose, but if the flood water still rose in the reservoir, the valve opened still more, and the gate gradually lowered again until with a maximum flood rise of 12 inches it was lowered to the full extent of 16 feet. It would be noted from *Fig. 20* that when the gate was fully open its crest formed an arc of a circle in continuation of the dam; the coefficient of discharge was therefore high, and experiments had shown it to be about 3·6. The automatic control had proved to be entirely satisfactory, and had already been dealing with floods of up to 20,000 cusecs at Pitlochry without any difficulty.

With 40,000 cusecs of floodwater discharging over the dam there was a very large amount of energy to be disposed of—about 100,000 kilowatts of power—and it had been decided that the best way to dissipate this was to throw it up in the air by means of a bucket on the downstream side of the dam, which had proved satisfactory.

Another work he would like to mention was concerned with the Fannich scheme (*Fig. 17*), where the Fannich tunnel was connected with the loch about 80 feet below the surface. The interesting point was that two shafts had been sunk on the line of the tunnel. That had been a great benefit, and he strongly recommended it to anyone with a similar problem. It had been intended in the first instance to leave a 60-foot barrier of rock between shafts to permit the tunnel, which was about 3 miles long, to be constructed quite independently of the work proceeding in the difficult portion under the loch. For various reasons it had not been possible to do that, and the contractor had been allowed to drive right through from the screen shaft to the gate shaft; before the blow-through was undertaken it was stipulated that he should therefore put in two concrete plugs while the work was proceeding under the loch. The intake tunnel was brought forward gradually until a 15-foot plug of rock was left where shown in *Fig. 17*, and a sump of sufficient size was provided to hold the 250 cubic yards of rock which it was anticipated would be blown out. Into that plug a hundred holes were drilled to about 2 feet from the bed of the loch. One or two holes were driven right through in order to check the actual thickness. About one ton of blasting gelatine was loaded into the plug, and the tunnel (up to the concrete plug) and the gate shaft were then flooded to within a depth of 12 feet below the water level of the loch, that distance having been very carefully calculated. The explosives were arranged in five separate sections, so interlinked that there would be no chance of a misfire. It had been calculated that at the peak of the explosion there would be a force of 10,000 tons on the concrete plug. For that reason another plug was put in downstream of the first so that if the first should fail the second would prevent the Loch from discharging down the tunnel. As a third precaution, a temporary steel bulkhead was erected as shown on the extreme left of the Figure.

The firing button had been pressed by Lady MacColl, and the explosion was a complete success; the water rose 35 feet in the gate shaft and just spilled over the top, as had been calculated, and then oscillated in the shaft for fully half-an-hour. Soundings taken afterwards showed that the anticipated amount of rock had been dislodged, and that it had fallen to a suitable level in the sump.

Mr J. K. Hunter proposed to deal with another aspect of hydro-electric development, namely, the benefits which were to be achieved by the integration of reservoir operation.

In general, river-basin development depended on the construction of a group of power-stations which were electrically interconnected, and the degree of regulation which could be afforded by the associated reservoirs would vary widely from one station to another. The maximum use of water would, under those conditions, require that the method of operation and the daily load factor of each station should be regulated in accordance with the prevailing conditions of flow. That meant that in wet weather the run-of-river stations would run on a block load, because they had no appreciable storage immediately behind them, whilst the output from the heavily-reservoired stations would be curtailed in order to reduce the consumption of stored water and they would carry the peak load. The reverse, of course, took place during dry weather.

The Scottish water-power schemes of which the Author had spoken were characterized by relatively incomplete regulation. The Author had already pointed out that the topography of the country did not permit the construction of really large-scale reservoirs, and the best had to be done with such regulation as was economically feasible. The operations of the Scottish Hydro-Electric Board were greatly assisted by the capacity of the National Grid to absorb much of the output of their stations as and when it became available; in other words, they had at their disposal a bottomless pit into which they could cast all their surplus energy. That was true, of course, only within certain limits which were set by the operational requirements of the thermal stations. It was, however, broadly correct.

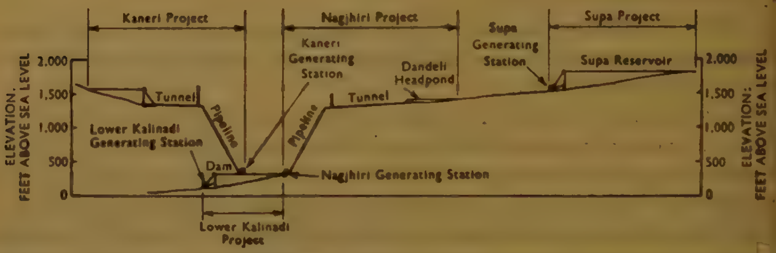
The integrated operation of reservoirs was particularly important where hydro-stations had no backing from steam stations. It might be of interest to refer to an example drawn from India, where plans had been prepared to develop the massive water-power resources of a portion of the Deccan plateau to the south of Bombay, where over a period of roughly 3 months in the year the south-west monsoon caused an extremely high rate of precipitation.

The problem there was to deal with the great seasonal variation in the run-off, 90 per cent of which took place in 25 per cent of the year. During the dry weather, the flow of the rivers dropped to insignificant proportions: any scheme of development was, therefore, dependent upon the possibility of constructing really large-scale storage.

Behind Goa on the Western Ghats was the Kalinadi River, where it had

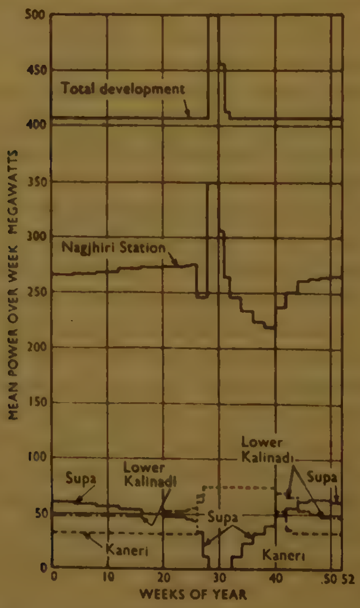
proved possible to store the run-off from about 400 square miles of catchment area by means of a very large reservoir which could, if required, be extended to store from 1½ to 2 years' run-off. By means of this reservoir

Fig. 21



KALINADI HYDRO-ELECTRIC SCHEME. DIAGRAMMATIC SECTION SHOWING PROPOSED METHOD OF DEVELOPMENT

Fig. 22



AVERAGE YEAR OPERATION OF KALINADI DEVELOPMENT. OPERATION DIAGRAM FOR COMPLETE DEVELOPMENT

it was possible, even in a dry year, to produce 1,800 million units in a single power station. *Fig. 21* showed diagrammatically the relation between a group of four stations which it was proposed to construct. On

the extreme right of the diagram was the principal reservoir—the Supa Reservoir—which incorporated a power station. The largest station in the group, Nagjhiri, would utilize the regulated water released from the Supa Power Station and would benefit from the monsoon run-off from a considerable proportion of unreservoired area. Lower down was a third, the Lower Kalinadi station, which was largely unregulated. The fourth station drew its water from an important tributary, the Kaneri, which could be only partially reservoired. Considered in isolation, the Kaneri development was not attractive, because it was not economically practicable to control more than 50 per cent of the annual run-off. However, by carefully planning the release of water from the various reservoirs and adjusting the output from the individual stations of the group it was possible to obtain what was virtually a uniform output not only throughout the year but from one year to another.

The lower part of the diagram in *Fig. 22* showed the operation of the four component stations week by week over the year. The Kaneri power station—the one of the four which it was most difficult to regulate effectively—would operate during the dry weather on stored water at an average daily output of about 33 megawatts. With the onset of the monsoon, the output from that station would be increased as the natural run-off increased until, when the reservoir was filled, it would continue to operate at its full output of 75 megawatts; then when the run-off dwindled the output from the power station would be gradually reduced, and eventually it would return to its dry-weather regime. The output from the other power stations would be adjusted correspondingly. The largest station in the group, Nagjhiri, was shown in the centre of the diagram, and in magnitude completely overtopped the others.

By thus co-ordinating the operation of the four power stations it was practicable, as Mr Hunter had remarked before, to obtain a uniform output of more than 400 megawatts of continuous power. The diagram in *Fig. 22* took no account of the variations in the daily load, nor of the variations over the week, but merely ironed out the short-term variations and showed the general picture.

Mr James Williamson referred to the diagram in *Fig. 8* dealing with the cost of tunnelling, which gave the cost per lineal yard. He would like to know whether the Author intended that diagram to indicate the cost of an excavated tunnel only—an unlined tunnel—or whether it referred to a completely lined tunnel. From the scale of cost per lineal yard, which appeared on the left-hand side of the diagram, it appeared that the minimum cost shown on the graph was about £80 per lineal yard, which Mr Williamson thought was rather a high figure. However, the Author would no doubt have some explanation to offer.

Mr Williamson said that he would like to compare that minimum of £80 with the actual cost of three lined tunnels in the Galloway scheme. The Loch Doon tunnel was a completely lined tunnel with a diameter of 8 feet

3 inches, and the finished cost had been less than £24 per lineal yard, as against the figure of £80 indicated by the graph. The cost of the Glenlee tunnel, which was 11 feet in diameter, had been less than £33 per lineal yard, including lining. Even on a short length of lined tunnel of 20 feet diameter the price had been only about £80 per yard.

Those figures were pre-war. One of the greatest disappointments in trying to carry out the Board's programme of work was that tunnelling difficulties arose to a very great extent and put up the costs, though it had to be said that, in the later stages, when the tunnelling gangs had got together and were working as willing teams, helping each other, and had gained experience, the prospects were very much better. No doubt some Members would have noticed the record rate of driving a small tunnel for the Loch Sloy scheme which had been mentioned in the Press. It had been claimed as a British record—426 feet in one week from one heading. He would like to mention that more than half of the crews were not British, but were displaced persons from Europe.

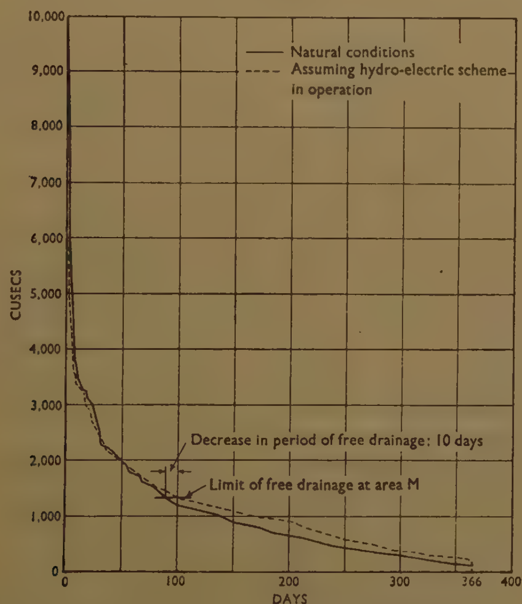
It was interesting to note what could happen after a world war, when workmen had had to change their jobs and were unsettled. The men had been for a long period in sheltered jobs and were not very willing to come to a place such as Sloy, where they found that it rained for about 3 days out of every 5. By about June, 1947, a labour force of about 1,400 men had been built up, but then several things happened all at once. There had been rumours of a " freezing " of labour—that men might be directed to stay on their jobs. There had been some bother with the camps and disputes about joiners, and there had been one or two mischievous people among the men. The result had been that there was great unrest and discontent, and after the Glasgow Fair holidays in July a great many of the men had not come back. In July, August and September the number of men had dropped from 1,400 to 900, and only from then on did a gradual build-up take place again. It could be reckoned that, owing to the loss of those 500 men, the completion of the work would be delayed by at least 6 months, which implied much additional cost. It was not until some of the strikes and labour difficulties had been overcome that the labour force had gradually been built up again, and a better spirit introduced. That was a matter which especially affected the tunnelling operations, because the loss of labour had taken place at a time when an effort was being made to get the main tunnelling work properly started.

On the Loch Sloy scheme in particular, the Author had referred to the collection of water and had given some indication of the value of gathering water from high-level catchment areas. Mr Williamson would like to express the value of the Loch Sloy scheme in a way which the layman could understand. The water from the catchment area of Loch Sloy was gathered from about 30 square miles of Highland country, with not a single inhabited house in that territory and not a single cultivated field ; there were only some sheep and some deer. The annual value of the ground

might be taken as 1 shilling per acre, or even less. The annual value of the power crop turned out from the mill at the side of Loch Lomond was equivalent to not less than £25 per acre of catchment area.

Mr Frederic Newhouse said that he proposed to deal with a point which the Author had mentioned on p. 258, namely, the fears of farmers and land-owners of damage to their land in the lower part of valleys into which flowed discharge from the turbines of power stations. The effect of a hydro-electric scheme on the water levels was, of course, that during flood

Fig. 23



FLOW DURATION CURVES FOR 1948

time the levels would be reduced, whilst during the summer they would be somewhat increased. Fears had been expressed that that might damage the crops, but it could be ascertained what the effect on the drainage would be in any particular instance.

Fig. 23 was a diagram which had been made to find out what the effect would be in the case of a scheme in Perthshire. A point M had been indicated in the lower part of Strath Earn as the criterion for the drainage of that area ; that was to say, nowhere would the damage to drainage be greater than on that land. The full line on the diagram represented the ordinary flow/duration curve for that river for the year 1948—the only year for which full observations were available at that time. The broken line

was the flow/duration curve which would apply, it was hoped, when the scheme was in operation. From that it could be seen what the decrease in the period of free drainage was. At present there was no drainage from the area M for about 90 days in the year, and when the scheme was built that would be increased to 100; which meant that for an extra 10 days a year the drainage of that area would be interfered with. Those 10 days, however, would not occur consecutively, but would be scattered throughout the year. Some of them would be in the wet part of the year and a few of them during the summer, so that in actual fact there would never be more than a day or two at a time when the drainage was interfered with, and it could therefore be said that the people in that instance had very little to fear.

Investigations of those difficulties, however, had shown that each scheme had to be considered individually. It was not possible to lay down a general rule or to derive a formula of any kind, because so many variables had to be considered in working out the two flow/duration curves—particularly the second one.

At the point M, a little more than half the flow of the river was controlled. The other half came from streams which entered the Earn below the last power station of the scheme in question, and therefore it was doubtful whether any further increase in reservoir content of the existing scheme could make much difference in that period of 10 days. It might increase it by a day or two, or it might not, but the limit of usefulness had been reached, and that was something which could be determined for any one place fairly accurately, so that it was possible to state for any definite scheme the damage that would be done to drainage.

The Author had suggested that the time had come to consider remodelling the river. That would be an extremely dangerous thing to do, because unless the river were considered as a whole remodelling might do more harm than good. It required, in Mr Newhouse's view, a study of several years, taking discharges under various conditions, before it would be possible to make a scheme for remodelling any river or any section of a river.

Another point which should be mentioned in that connexion was that the people who were cultivating land in the area were cultivating on the flood plain, and naturally they should expect to be flooded. That happened all over the world, and it could not be avoided. Farmers in the Earn and many other flood plains used to try to protect themselves by means of banks, but in latter years the banks had been neglected and breaches left unrepaired—presumably because repairs had been considered to be uneconomical. If sufficient banking had been built to keep the floodwater off the land entirely, there would be more water in the river lower down and difficulties would arise there. Probably the best plan was to let them continue as they were and to pay for any damage which was done.

The Author had remarked that it was curious that power schemes nearly always produced more power than had been originally estimated.

That was partly due, no doubt, to the very conservative estimates made by the Consulting Engineers, but it might also be attributed to two other reasons. The line on the diagram (*Fig. 23*) for the limit of free drainage, if extended to the left, could be called the utilization line for any scheme, above which it would not be economic to make aqueducts and reservoirs to hold any more water. There would be an area above the line between the axis and the black curve where the water would theoretically be wasted. As a matter of fact, however, not all that water was wasted, because some of it arrived at a time in summer when the reservoir was not full; some of it could therefore be used, although it had not entered into the calculations which had been made. The other point was that the Meteorological Service had, he thought, underestimated the rainfall on the higher levels in Scotland. He believed that most people who had dealt with run-offs of the streams and compared them with the rainfall would be inclined to agree on that point.

Mr J. A. Banks referred to the Author's statement, under the heading "Design and Constructional Methods," that there were fewer examples of novel civil engineering design in Scotland than in other countries. If that were so, it was not due to any lack of ingenuity on the part of the engineers, nor did he think that it had been the Author's intention to suggest that it was. The French had certainly shown commendable ingenuity with their ski-jump spillways, their power stations curved in plan, and their dams of spherical form, but those designs had been forced on them by the narrow gorges to which the Author had referred, particularly in the Massif Central, and it was doubtful whether the same schemes could economically be made applicable to the usual type of valley in Scotland, which was much wider and where the same difficulties did not arise.

On the question of dams, the Author had referred to the rate of 5,360 cubic yards of concrete placed in a week on the gravity dams at Affric, regarding that as a reflection on the more difficult formwork necessitated by the buttress type. Mr Banks thought that that figure was liable to be somewhat misleading, because in the normal round-headed or diamond-head buttress type the total mass of concrete was only about two-thirds of what it would be in the gravity dam, which would mean that in effect a pouring of 3,500 cubic yards per week would show comparable progress. The shuttering question should not be over-emphasized, because the cost of shuttering was only a small percentage of the whole, and even if the cost were double it would have no great material effect, if there were a saving in concrete such as usually resulted. The relative cost would vary on different sites, but it was of some interest that on the Shira site the ratio of the cost of the round-headed buttress type to that of the gravity type was 0·77 : 1. In that particular case, the change-over line was at 55 to 60 feet, below which height the gravity dam would have economic advantages, whilst above it, the buttress type would be more favourable.

He would like to say a word about the Trief process, to which the

Author had referred, and which was used in France. It was of interest to know that the Hydro Board were going to construct a sizable dam by this very impressive process in order to try it out on full scale. The most suitable slags available in Scotland were already being used for the production of blast-furnace Portland cement. Other slags were deficient in lime as compared with the French slags. It would be highly creditable if those difficulties could be overcome.

One point which was not mentioned in the Paper, but which was of importance economically, was the question of handling cement. An engineer coming from any one of many countries abroad would certainly be astonished to see the handling of hundreds of thousands of tons of cement in bags on jobs in Great Britain. With bags costing 20 shillings per ton of cement, no pride could be taken in the lack of bulk-handling facilities in Scotland; such facilities could effect a great saving in cost with less waste of cement, and would be an all-round advantage.

He would like to add to the Author's reference to tunnelling and to the nature of the materials. The main Shira tunnel was in phyllite, which was a soft schist-like materal, but only 5 per cent of support had been necessary in a 4-mile length of tunnel, though it had been thought that possibly 12 per cent of support might be required. The inclined pressure tunnel required no support at all, being in a hard schist. That pressure tunnel had been excavated at an angle of 39 degrees to the horizontal.

On the subject of tunnel costs, Mr Banks agreed with Mr Williamson, being of the opinion that the costs shown in *Fig. 8* were on the high side. Also he thought that 8 feet was not quite the economic minimum size; diversion tunnels of a smaller size than that had been constructed quite economically. Contractors' prices for aqueducts were very variable, but a 42-inch-diameter pipe was about the economic limit on the job at Shira to which he had referred. If the capacity required exceeded that of a 42-inch pipe, then a trapezoidal channel was warranted.

Reference had been made to the reverse pumping on the Shira scheme. That was an interesting matter and had particular application in that case; a relatively large proportion of water would be diverted into the lower dam, and, since the lower dam had very much less capacity than the upper one, there was risk of excessive spill if some such expedient were not adopted. By pumping an average height of about 130 feet into storage in the upper reservoir, water would be conserved for two-stage generation through a head of about 1,100 feet.

Dr Charles Jaeger observed that the Scottish schemes were remarkable for the great length of aqueducts for water diversion to the main reservoir and for the importance of the diverted discharge. That was to some extent a very different technique from earlier continental practice, by which only one valley was equipped with one large power station or a cascade of power stations. It might be noted, however, that other countries had recently been converted to an extensive use of diversion aqueducts. He could

mention Italian and Austrian schemes, but the most interesting examples were provided by certain Australian schemes and by a scheme in Switzerland.

The Australian schemes were well known in Britain, but he would like to mention in particular the Snowy Mountain scheme in New South Wales, the Kiewa project in Victoria, and the scheme at Wayatinah in Tasmania. In Switzerland, the Grande Dixence scheme was to be noted. The dam of the Dixence reservoir was to be replaced by a far larger one, and the extra storage so created would be filled afterwards by an extensive network of aqueducts at high level, some of which would start directly under the glacier. The reasons why diversion aqueducts were now being used in many countries were the necessity of concentrating water in a few very large storages and of taking advantage of the great heads which geological and topographical conditions allowed to be developed.

Referring to the buttress dam, Dr Jaeger wished to know whether any comparative figures were available on the volume of concrete required for the three typical solutions shown in *Figs 7* of the Paper. He knew that photoelastic research had been carried out in order to solve the problem of stress distribution in the head of the buttress, and he would like to know if and where the results had been published. In connexion with the buttress dam of medium height, he would like to mention the very thin multiple-arch buttress-type dam. That type was popular 20 years ago in other countries, but recently Electricité de France had reverted to a very thin multiple-arch dam design for one dam in the Massif Central (Faux-la-Montagne Dam). This was a replica of the old Swiss dam at Les Marécottes, built 20 to 25 years ago. The French dam was on a larger scale and was designed by a French contractor in Toulouse. The dam at Les Marécottes had been in operation for more than 20 years, and no cracks whatever had been observed. That solution might be worth further study.

Mr P. O. Wolf referred to the section in the Paper on the value of diverted water which, together with *Fig. 5*, presented a rigorous method of determining the economy of diversions and the size of diversion works. He strongly advocated the application of such rigorous methods of analysis, in place of " rules of thumb " which usually fitted only one set of conditions and one area.

Mr Banks had already discussed the Author's reference to the very large quantity of concrete—5,360 cubic yards—placed in one week in a mass dam at Mullardoch. At Loch Sloy dam,[1] which was of the buttress type, the peak rate of concreting apparently exceeded 4,000 cubic yards per week. Both those outputs were the results of the excellent utilization of economical resources of labour, plant, and materials, and they represented about equal progress in reservoir construction.

With reference to the remark on p. 261, that schist could not be used as

[1] James Stevenson, " The Construction of Loch Sloy Dam." Works Construction Paper No. 20. To be published in Proc. I.C.E., Part III, vol. 1, Aug. 1952.

a concrete aggregate, Mr Wolf pointed out that the term schist covered a range of rocks, some of which were definitely unsuitable, whilst others proved, on test, to be acceptable. There was, for instance, hardly any other material available within reach of the site of one of the large dams mentioned in Table 5, and the coarse concrete aggregate consisted predominantly of a suitable schist.

The ratio of diameters of unlined tunnels to lined tunnels of equal carrying capacity was stated, on p. 261, to be 1·40, and an excess excavation of 96 per cent was postulated for unlined tunnels. Those figures represented medium-sized tunnels but would vary according to size and to the roughness of rock. For small tunnels, for instance, the excess excavation might be as low as 40 per cent, and in that case lining of the arch would probably not be economical.

In addition to the fish passes mentioned in the Paper the artificially roughened chute mentioned in the Institution Report on Fish Passes should be borne in mind. That might be of particular value in reservoirs with a large range of water level where a hinged chute projecting upstream from the dam automatically adjusted itself to variations in upstream level.

Mr A. C. Allen observed that the Author had mentioned that there were one or two recognized earthquake zones which it would not be possible

Fig. 24

Comrie Earthquake Zone

to avoid entirely. One earthquake zone in Scotland was around the village of Comrie in Perthshire, as shown in *Fig. 24* in relation to some of the proposed works of the Breadalbane Hydro-Electric-Scheme, which was at present under promotion.

The earthquakes were caused by movement associated with the Highland Border fault, which outcropped on the line shown in *Fig. 24*. Although not apparent from geological evidence in the district, it almost certainly headed to the north-west, so that it lay under Comrie at a considerable depth.

The Comrie earthquakes had been carefully chronicled since the year 1788 and were remarkable for their extreme localization. In six of those recorded, the position of the centre of the disturbance, or epicentre, was known with some accuracy, and lay within $1\frac{3}{4}$ mile to the east and a mile or so to the north-west of Comrie, so that the extreme distance between epicentres was little more than 2 miles.

The most active period known to date was during the 10 years 1839 to 1848, when no fewer than 320 shocks were felt in Comrie. Of these, three were principal earthquakes, and two of them reached an intensity of 8 on the Rossi-Forel scale. That intensity was defined as being "very strong" and corresponds to chimneys and walls being thrown down and cracks appearing in the walls of some buildings. In the more severe of those two earthquakes, that intensity had been recorded at distances of up to 6 miles (see circle in *Fig. 24*) from the supposed centre of the earthquake.

The magnitude of the inertia force acting on a mass during an earthquake was usually expressed as a proportion of the force of gravity acting on that mass, that was to say, a proportion of its weight. A number of students of earthquake action had proposed that for a force of intensity 8 that proportion should be between 0·05 and 0·10. The force acted in a vertical direction over the epicentre of the disturbance and inclined more to the horizontal the further out from above the epicentre it acted.

The proposed Glen Lednock dam would be 5 miles to the north-west of Comrie, and, if the dam was an earth-fill or rock-fill design with a flexible core-wall, it was unlikely that any damage would occur to the dam provided that earthquakes in the future did not exceed those which had occurred in the past; but that could not be said if the design of the dam was of a rigid type, such as a mass-concrete gravity design.

The following figures referred to a gravity dam 100 feet high and of simple triangular section. The density of the material of the dam had been taken as 150 lb. per cubic foot, and the uplift had been measured equal to two-thirds of the hydrostatic pressure at the heel, reducing uniformly to zero at the toe. Without taking earthquake effects into account, and using conventional methods of design, the base width required in order to eliminate tension at the heel, with a full reservoir, was 76 feet, and the corresponding sliding factor was 0·76. When earthquake forces due to an acceleration of 0·05g were included in the design, the base width required to satisfy the same criterion was 82 feet, resulting in an increase in the volume of the dam of 8 per cent. The effect of such earthquake forces on a dam not designed to resist them would be to induce tension in the upstream face and to increase the sliding factor from 0·76 to 0·87.

Although there had been no such earthquakes as those described for just over 100 years in Comrie, it was not an assurance that they had ceased, and minor shocks had continued to be felt. In view of the serious damage and loss which would follow the failure of the dam, impounding an immense volume of water and located as it was, and also in view of the uncertain nature, magnitude, and duration of earth tremors, it would seem that this was an instance where earthquake forces should be included in plain gravity and buttress designs. It would be of interest if the Author in his reply would say a little more about the consideration which had been given to earthquakes, and would expand on his remark that any earthquake movements were likely to be so small that they were being treated as of minor significance.

Mr Robert Carey referred to what the Author had said on p. 253 of the Paper, on the question of new designs. Like Mr Banks, he did not think that the Author intended to cast any slur on the engineers concerned, but it should be said, in justice to the engineers connected with the work, that some of those new ideas had been investigated with regard to Scottish works. For instance, rock-fill dams with thin core-walls had been investigated, but with a dam of that description it was necessary to have a proper selection of fine filling upstream to form the water barrier when the thin core-wall deformed owing to the settlement of the dam ; such fine material had not been available in any of the schemes with which Mr Carey had been connected, so that that type of dam had been ruled out. Similarly, underground power stations had been considered, but there again physical difficulties arose, such as those mentioned by the Author later in the Paper, and the difficulties of access in the Scottish hills. Another point which weighed very heavily against underground power stations was the cost of rock tunnelling and rock excavation.

One factor which might induce reconsideration of the underground power station was the shortage of steel and cement. A great deal of steel was necessary for, say, a high-pressure pipeline or tunnel and, if the power station could be put underground near the dam, the result might be that one would only have a tail-race tunnel of perhaps larger dimensions but with no lining, and a considerable saving in steel and cement would be achieved. It might be necessary to contemplate a little extra cost in the case of those power stations in order to save material.

Fig. 19 was a diagram of the Affric-Fasnakyle tunnel. It might be worth noting that the high-pressure section of that tunnel had originally been designed with a reinforced-concrete lining which took the full internal pressure, and a thin steel watertight membrane. After further investigation it had been found that the rock was a very sound psammitic schist and impervious, and it had therefore been decided to cut out the reinforcement and the steel membrane and to line the tunnel with concrete, with pressure grouting behind to fill up any fissures in the rock. The only lined part was a length of about 800 feet at the lower end, which had been lined with

heavy steel to take the full head where the tunnel did not have sufficient overburden of rock to take it. In the remainder of the tunnel, the rock took the full internal pressure and thus a considerable amount of steel was saved.

On p. 263, when dealing with screens, the Author had already corrected the reference to the $\frac{1}{2}$-inch vertical by $\frac{3}{4}$-inch horizontal mesh for smolts, but there remained the question of the spacing for salmon. In the Spey works of the Lochaber scheme, 2-inch spaces were allowed, but the Author referred to $1\frac{5}{8}$-inch, a figure presumably based on more recent experience than the Spey works. It was a small point, but it made a considerable difference to intake works where there was a very fine screen, particularly with such low velocities as 1 foot per second.

On the question of buttress dams, the Author had stated that they had an advantage over gravity dams in that they used considerably less concrete. Mr Carey would like the Author to clear up one point in that connexion. In Table 5 (*a*), for the Mullardoch concrete gravity dam, 275,000 cubic yards of concrete were shown, whilst for the Shira upper dam, which was a concrete round-headed buttress dam, the concrete required was given as 300,000 cubic yards. The Mullardoch gravity dam was higher and longer, according to the Table, yet it appeared to require less concrete than the buttress dam. Perhaps the Author could clarify that point.

*** **Dr W. L. Lowe-Brown** referred to the aqueducts which the Author considered to be the characteristic feature of Scottish hydro-electric practice.

Dr Jaeger had stated that aqueducts were being used extensively on several of the Australian schemes. That was only correct in one sense. On a recent visit to Australia Dr Lowe-Brown had had the privilege of being shown over the works of the Kiewa Scheme by the State Electricity Commission of Victoria. That was the next largest scheme in Australia to the Snowy River scheme but was in a much more advanced state of construction. On this scheme it was proposed to use more than 180 miles of very large-sized " race lines," as aqueducts were referred to in Australia. At that time, however, they only existed as lines on the map. In soft ground they were to be constructed by special mechanical plant but he did not believe that any had yet been built in rocky country. The Australian engineers were still trying to decide on the best and most economical design to adopt. The sites in which many of those designs were proposed were on the steep sides of rivers in rough rocky country where concrete-lined or unlined prismoidal-type conduits of large size would be extremely costly to construct. Fortunately, the climate was not so severe as in Scotland and, although there were frosts for several months every winter, it was not considered necessary to cover the race lines. He felt sure that all engineers everywhere would be pleased to obtain from actual experience in Scotland any information that would help them with that difficult problem.

*** This and the following contributions were submitted in writing.—Sec. I.C.E.

Mr F. W. Coates observed that the Author, in commenting on design and constructional methods used, had suggested that when the work was carried out by contract, as was current practice, there was less freedom for changes after the contract had started than there would be with a direct labour system. It ought to be emphasized that delayed alterations in designs were expensive, whatever system of construction was adopted, although the increases in cost might be more obvious under the contract system.

In discussing the form taken by hydro-electric developments in Scotland, the Author had stated that the annual output of any scheme varied little with the capacity of plant installed. That was generally true for schemes with large storage available, but with schemes forming part of a highly integrated " grid " system (so that the hydro-electric scheme did not need to adhere rigidly to a particular load curve), the less the storage available, the more important became the plant capacity in its effect on average annual output of units, until in the case of the "run-of-river" scheme, with no appreciable storage, additional installed plant could considerably increase the average annual output—particularly with the " flashy " types of rivers and streams.

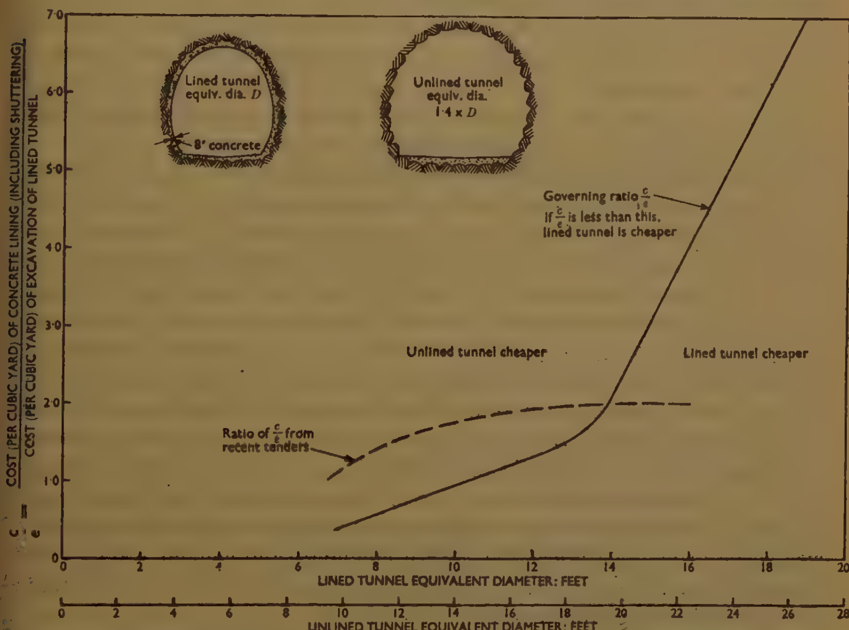
It might well be that the increase in output of the developed schemes beyond original estimates, which was referred to in the Paper, arose from the steadily increasing demand for power throughout the years. That might have had the effect of reducing the necessity for the scheme to adhere to the original load curve, and of creating a demand for units which might otherwise have been spilled.

Referring to Table 3, which showed the capital cost for impounding per unit stored for various post-war and pre-war schemes, Mr Coates pointed out that the attractively low cost of impounding for the Fannich scheme resulted from the fact that no dam had been constructed and, as stated later in the Paper, storage had been provided by lowering the existing level of the loch by 50 feet. He wished to ask if the Author could explain how the capital cost for impounding had been determined in that case. Was it merely the cost of the tunnel to the generating station, and if so, were the power tunnels for the other schemes similarly included in determining their capital cost for impounding?

In discussing tunnels, the Author had stated that it was still more attractive economically to line a tunnel with concrete than to use a larger unlined tunnel of equivalent carrying capacity. The choice between the two was governed by the ratio of cost of lining (including shuttering) per cubic yard to cost of excavation per cubic yard for different tunnel diameters. *Fig. 25* showed the governing value of that ratio for various tunnel diameters such that, if the ratio fell below the value given on the curve, the small-diameter *lined* tunnel would be cheaper. The curve was based on the excavation rates per cubic yard which were given by the Author on *Fig. 8*, and the ratio of unlined to lined equivalent diameter of

1·40 as suggested in the Paper. The lined tunnel was assumed to be of horse-shoe section with an average 8-inch thickness of 4 : 1 concrete lining for all diameters. The unlined tunnel was also assumed to be of horse-shoe section and had an invert lining 8 inches thick for ease of access. Without that invert lining the unlined tunnel would be still more attractive economically. As might be expected, the governing ratio of c/e increased with increasing diameter. That gave more opportunity for the lined tunnel to be the more economical since, whilst excavation quantities

Fig. 25



COMPARISON OF COST OF LINED AND UNLINED TUNNELS OF EQUIVALENT CARRYING CAPACITY

increased as the square of the diameter, lining quantities increased only as the diameter.

Recent tenders had given values of approximately 2·0 for the ratio c/e, and it could be seen from *Fig. 25* that, for this value, the fully-lined tunnel was the most expensive for a lined diameter of less than approximately 14 feet.

In the future, no doubt, the use of pneumatic concrete-placers, such as the American " Pressweld " type, would reduce the cost of lining, but with present methods of construction, if cost were the only governing factor, the

unlined tunnels would appear to be the most attractive proposition for the smaller diameters.

Mr C. M. Roberts observed that it had been originally decided to construct the Mullardoch dam to a spillweir crest level of +817·00 O.D. so as to form the main storage reservoir of the Affric scheme. In that way, the reservoir formed on the River Affric would not flood back to Loch Affric and thus detract from the beauty of the area. At the beginning of 1950, however, when the Mullardoch dam was partially constructed, it had been decided to reduce expenditure on that portion of the works, in order to allow the construction of other works, by reducing the crest level of the south wing of the dam by 20 feet. The maximum economy had been effected by stopping all blocks under construction in the south wing at a level of +797·00 O.D. and by reducing the remaining blocks, including those just started, to a section appropriate to the lowered top-water-level. The width of foundation required for a crest level of +817·00 O.D. had, however, been excavated and filled with concrete to allow for future raising.

In 1951 it had been decided to complete the dam to its original design crest level while the civil engineering contractor was still on the site, thereby taking advantage of the plant already in position. That was being done by placing a concrete backing slab on the blocks of the dam which had been reduced in section. To allow for the relative movements caused by the temperature differences between the concrete already placed and the new concrete, and by the plastic flow of the concretes, the backing slab of each block was, in the first place, bearing on the old concrete only by way of three pre-cast concrete ribs with greased building paper between contact faces, a 3-foot-wide slot having been formed elsewhere between the backing slab and the main block of the dam. The face of the old concrete was scabbled and the inside face of the new concrete was keyed so as to ensure a thorough bond between the concretes and the slot filling, which was to be of aggregate grouted with a colloidal sand-cement grout. The grouting operation would be carried out when the relative movements decreased to a negligible amount.

Referring to the typical flow-operation curve for a " flashy " stream, in *Fig. 6* of the Paper, Mr Roberts said that he would like to ask the Author whether that was based on actual records obtained in the Scottish highlands or on assumption. The curve was of considerable importance because there had so far been a dearth of such records for streams in Scotland, and it would be interesting to know whether that curve could be taken as a reliable guide for streams of that category.

Mr Carey had raised the question of the advantages of adopting underground power stations with long unlined discharge-tunnels, and had pointed out that there would be a saving in steel and cement. Mr Roberts would like to add that a decrease in the period of construction of the tunnel could also be expected, because the concrete lining and grouting operations, except in zones of weak rock, would be eliminated. The effects of the

saving in tunnelling time would, however, have to be related to the constructional time required for the scheme as a whole before the extent of the advantages could be appreciated.

Mr William Allard asked whether the Author could state the relation between the 1943 estimate of potential power in the Highlands, which amounted to more than 6,000 kilowatt-hours, no account having been taken of current developments at that time, and the estimate [1] made in 1921 by the Water Power Resources Committee, for the whole of Scotland, but again excluding current developments.

The transfer of water in many places from catchment to catchment, though probably not the whole flow of any particular area in most cases, presented a discouraging prospect from the point of view of hydrological studies, unless there was to be a volumetric measurement at each of the numerous transfer points.

The Author, in reply, expressed his gratitude for the reception given to his Paper and said how glad he was that Mr Donkin had found it possible to be present and to pay such a well-merited tribute to the late Sir Edward MacColl, who had been, of course, the mainspring of the Board's activities. The Author was also glad that so many of the engineers concerned had taken this first opportunity of filling in the blanks in his Paper. That was what he had hoped would happen, and it would be excellent if they could follow up with their own Papers on some future occasion.

Mr Hunter had referred to the use of storage and had compared the conditions in Scotland with conditions in India, stating that in Scotland they had a " bottomless pit " into which to deliver all their surplus units. It was perhaps an exaggeration to say that it was a bottomless pit. Steam power generating engineers did not like to shut down their high-performance steam plants to leave room for some water-power units.

Mr Williamson had asked whether *Fig. 8* referred to excavation only. It did, and the cost of lining was not included. It was based on up-to-date figures. The Author was not surprised that both Mr Williamson and Mr Banks felt that the figures were on the high side but costs were still rising. The comparisons made by Mr Williamson between Galloway tunnelling costs and those in *Fig. 8* simply reflected the considerable inflation which had taken place since the early nineteen-thirties, and hydro works built then were now very profitable. The inflation had been such that schemes which had been in the borderline category before the war were now able to produce energy at about one-third the price at which it could be produced today at a post-war steam station.

Mr Newhouse had dealt in some detail with the problem of interference with drainage margins. That was a question which was bound to arise often, because the more that a watershed was reservoired the more the flow over the year was averaged out. People who had been accustomed to

[1] Final Report of the Water Power Resources Committee of the Board of Trade. H.M.S.O., 1921, p. 35.

a low flow in the summer did not like it when the summer flow kept up at a higher level than that to which they had been accustomed in the past. The remodelling of rivers to improve drainage was not being advocated by the Author. That was really a matter for the Department of Agriculture. In many parts of Scotland they were, he thought, looking at that problem a great deal, and he only hoped that anything they did would be benefited by any improved regulation.

Both Mr Banks and Mr Carey had commented on the observation which had been made about the absence of novel designs. It was no reflexion on the engineers; it was simply that the conditions which had ruled in Scotland had not given as much scope as was found in some other countries.

Mr Banks, and later in the discussion Mr Wolf, had quite rightly pointed out that in comparing concrete-placing rates regard should be had to the ratio of dam height to bulk. Although agreeing on this point the Author was not satisfied with Mr Banks's remark that the shuttering question was of minor importance. It was difficult to persuade joiners or carpenters to go to the remoter Highlands and without them progress could be so seriously upset as to make an attractive proposition on paper quite the reverse. That point was emphasized in the second last sentence of the Paper and made the Author reluctant to accept any general proposition of a fixed change-over line from one type of dam design to another. For similar reasons the Author would hesitate in agreeing with Mr Banks that a 42-inch-diameter pipe was about the economic limit for piped aqueducts. Any addition in on-site formwork involving the use of skilled workmen simply increased the risk of delays.

The Author was very glad that Mr Banks had mentioned bulk handling of cement and the amount of money which was thrown away every year in cement bags which could not be returned because they were opened with a spade or left out in the open with cement clinging to them. It would be very useful indeed if bulk deliveries of cement could be organized for works such as those in the Highlands.

It had been very gratifying to hear from Dr Jaeger that many other countries were now following the example of Scotland and collecting water from adjacent catchments in much the same way as had been done in Scotland. It simply meant that no country could afford to ignore any water which could be economically collected.

Dr Jaeger had asked if comparative figures for volume of concrete were available for the dam designs shown in *Figs 7*. There was very little difference in the amount of concrete in the round-headed and the diamond-head buttress designs. Concrete in the massive-buttress design, however, was greater in volume by nearly 20 per cent. Dr Jaeger had also asked whether any information was available about the stream-distribution experiments which had been made on some of the dam designs of the buttress type. Abridged results had been given at a meeting in April 1951 of the Experi-

mental Stress Group of the Institute of Physics, Liverpool. It was the sort of information which the Board desired to disseminate, so the Author hoped that Mr Guthrie Brown, of Sir Alexander Gibb & Partners, who had organized the tests, might find an opportunity to publish them before long by means of an Institution Paper.

The only hesitation which the Author would have in accepting Dr Jaeger's advocacy of a thin multiple-arch dam design would be in obtaining adequate security against the aggressive action of the acid waters which are universal in the Scottish Highlands. A thin shell did not provide much margin for losses, which, in the case of unprotected concrete, could amount to as much as an inch in a few years.

Mr Wolf was quite correct in drawing attention to the wide range of schist rocks which could be encountered. It would have helped if, in referring to schist rocks being unsuitable as aggregates, the term had been prefixed by the word "mica." However, it might be of interest to record the rocks used, or to be used, in constructing the dams listed in Table 5. They were:

Sloy	Epidiorite
Pitlochry	Quartzite
Clunie	Quartzite
Errochty	mainly Quartzite and some Granulite
Mullardoch, Benevean	Psammitic Granulite
Shira	Epidiorite

The Author was unable to follow Mr Wolf's conclusion that the excess excavation for small tunnels might be as low as 40 per cent. The probability was that the roughness of an unlined tunnel would be relatively greater the smaller the diameter whereas in a lined tunnel the coefficient of friction was not likely to vary so greatly with size. On this reasoning one would expect the excess excavation for small tunnels to be greater than for medium or large tunnels.

Whilst the use of an artificially roughened chute type of fish pass might have an application in the case of reservoirs with a large range of water level, it had so many other drawbacks that it would, in the Author's view, be better to develop a design of Borland pass to suit such conditions. That was in fact being done and at a dam with a range of 40 feet a variation of the Borland design was to be used.

It had been a great pleasure to find a Student making such a useful contribution to the discussion. Mr Allen had quoted some very interesting figures. The earthquake zone at Comrie was one of those which the Author had had in mind in his reference to earthquakes on p. 252 of the Paper. Available geological records supported a probable intensity of 8 on the Rossi-Forel Scale. It was an intensity which, as Mr Allen had demonstrated, could be readily provided for if the dam were to be of concrete

gravity design, and virtually ignored, if, as was likely, the dam was of earth or rock-fill construction.

The Author had already commented on Mr Carey's remarks about the absence of novel designs in Scottish hydro-electric works. His illustration of what would be involved in putting a power station underground was most useful and agreed with the Author's own views.

The Author agreed generally with Mr Coates when he stated that delayed alterations in designs were expensive, but occasions did arise when some substantial simplification could be effected. The full monetary value of those was difficult to obtain under the contract system.

The reference in the Paper to the annual output varying little with the capacity of plant installed was true, as Mr Coates had pointed out, for schemes with storage. In the case of run-of-river schemes the output increased markedly with increased capacity, but the cost of the extra units could be quite uneconomic. Mr Coates's theory that the increasing demand for power might be responsible for the output of developed schemes being greater than the estimates would really only apply to run-of-river schemes. For schemes generally the explanation was most likely to be found in the conservative estimates supplied by the Meteorological Office of the rainfall to be expected on the catchments concerned.

In Table 3, only the cost of providing the underwater intake works was allowed for in arriving at the capital cost of impounding at Fannich. No tunnel costs were included either there or in any of the other cases quoted.

In *Fig. 25*, Mr Coates had shown that for lined diameters of less than approximately 14 feet the fully lined tunnel was the most expensive. The answer depended, of course, on the comparative tender prices used, but even so the kink in the governing ratio $\frac{c}{e}$ curve in his Figure was difficult to understand. The figures available to the Author and upon which he had relied in his Paper did not give the results which Mr Coates showed in *Fig. 25*. Instead, for the whole range of tunnel sizes from 8 feet to 20 feet, the extra cost per lineal yard for rock excavation, as an alternative to lining, ranged from + 3 per cent to + 7 per cent of the cost of the tunnel.

Mr Roberts had asked whether *Fig. 6* was based on actual records. Although the curve was described as typical it was actually based on gaugings taken on the Allt Uaine, a small stream with a catchment of 1·2 square mile, feeding Loch Sloy. *Fig. 6* could therefore be taken as applicable to any streams of a similar nature.

In reply to Mr Allard, the estimate made in 1921 for the whole of Scotland by the Water Power Resources Committee amounted to 1,880 million units or 217,000 kilowatts continuous, exclusive of completed developments at that time, which were small. The estimate made and published in their Development Scheme by the North of Scotland Hydro-Electric Board in 1944 was 6,274 million units (715,000 kilowatts continuous) for the North of Scotland area alone, and again exclusive of

current developments, which were then about 1,500 million units including the Galloway and Clyde Hydro schemes.

Mr Carey had asked about the permitted spacing between bars for salmon screens. No doubt the size of salmon varied from one river to another, but on the Board's works the provisions for fish were supervised by a Statutory Fisheries Committee, and in addition the Board retained the services of a very experienced Fishery Adviser. To take care of varying circumstances it had been decided to standardize on a clear space of $1\frac{5}{8}$ inch. If that could be increased later on to 2 inches, as on the Spey works, the Author would personally be very pleased.

The Author was grateful to Mr Carey for drawing his attention to apparent inconsistencies in the figures in Table 5 (*a*). Some adjustments were to be made in the Table before printing in its final form,* but it was necessary to point out that some of the dams mentioned in the Table were still under construction, so the quantities could only be estimates. Also, since the centre of the Mullardoch dam rested on an island a direct comparison on a total length and height basis was liable to be very misleading and illustrated how the shape of a valley could influence the amount of concrete used.

* These adjustments have been made.

Paper No. 5844

"A 75-inch-diameter Water Main In Tunnel: A New Method of Tunnelling in London Clay"

by

Peter Adamson Scott, B.Sc., M.I.C.E.

(Ordered by the Council to be published with written discussion) †

SYNOPSIS

To deal with the increasing demand and diminishing supply of water for east London, the Metropolitan Water Board decided to abstract water from the River Thames at Hampton and pipe it across London to their storage reservoirs in the Lee Valley, a distance of about 24 miles. A 75-inch-diameter main was required, and to lay this through the built-up areas of London would present almost insuperable difficulties. It was therefore decided that 19 miles of the main should be laid in tunnel, and to economize in the size of the tunnel required for laying such a large main, it was recommended that a new method should be adopted.

The proposed design involved driving a tunnel of a slightly greater diameter such that the steel main could be drawn through it, the joints being welded from the inside only, and the annular space between the pipe and tunnel lining being filled with concrete. An experimental 1,000-foot length of this type of tunnel was driven to test its feasibility, and a new method of tunnel-lining—a system of pre-cast unreinforced-concrete wedge-shaped sections—was adopted; the advantage of this system, when used in London clay, being that no steel is required for reinforcement or for bolts, and no grouting is required between lining and clay.

The new tunnel-lining system proved to be highly satisfactory, and it was established that steel pipes could be drawn along the tunnel lining with a very small clearance. Welding of joints and filling of the annular space between pipe and tunnel-lining were carried out successfully and economically.

The Paper describes the special plant and methods which had to be devised to carry through this experimental work.

Introduction

Faced with the problem of an ever increasing demand for water in east London, and a diminishing supply from the River Lee, the Metropolitan Water Board decided that their best solution would be to abstract water from the River Thames and pump it across London to their storage reservoirs in the Lee Valley, a distance of about 24 miles. To enable the reservoirs in the Lee Valley to be refilled with Thames water during a winter season, a high rate of pumping would be necessary, and the capacity

† Correspondence on this Paper should be received at the Institution by the 1st August, 1952, and will be published in Part I of the Proceedings. Contributions should be limited to about 600 words.—Sec. I.C.E.

of the main was consequently fixed at 120 million gallons per day. For this purpose a 75-inch-diameter main would be required, and the practical difficulties of laying a pipe of such size by means of surface excavation over a distance of 24 miles, most of which would be through closely built-up areas led to the decision to lay it in tunnel.

The firm of Sir William Halcrow & Partners was asked to study the problem and put forward a suitable design. It became evident at an early stage that the cost of building a tunnel large enough to permit laying a 75-inch main within it, would prove to be exorbitant. The inherent wastefulness of providing a large-diameter tunnel merely to accommodate a much smaller main has, in any case, always been considered objectionable by this firm, but the difficulties of alternative forms of construction, designed to avoid this waste, have hitherto prevented their adoption.

The problem which presented itself was to devise a method of laying and jointing a large-diameter pipe inside a tunnel of only slightly larger diameter, and subsequently filling the space between the main and the tunnel. Since the tunnel lining would be of no permanent value to the main, under such conditions, its cost should be kept to the absolute minimum. At the same time, since the main would be under pressure, the joints between pipes had to be completely watertight. Once laid, the external skin of the pipe would be inaccessible for maintenance and had therefore, to be proof against corrosion. The fact that the main could not, for economic reasons, be duplicated, necessitated the provision of internal protection of very high quality.

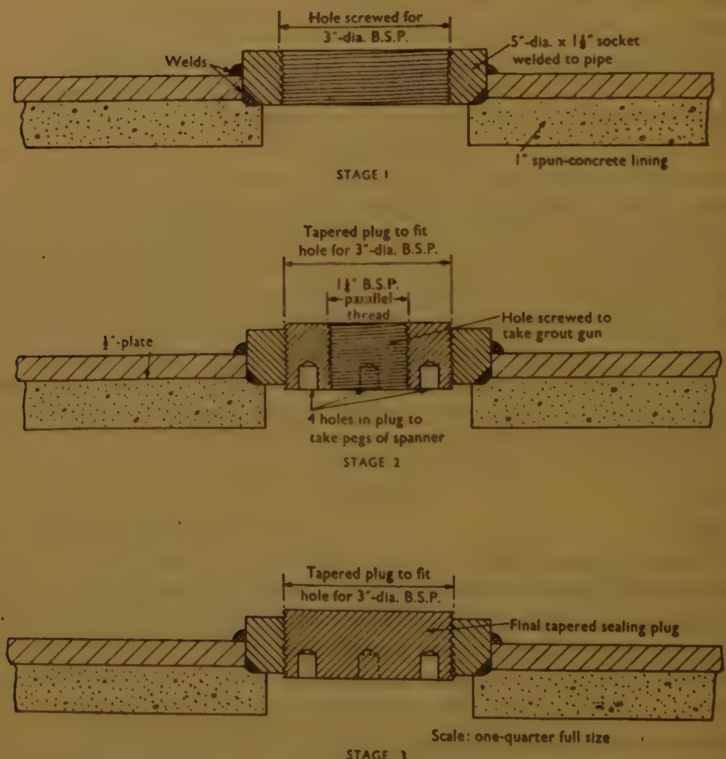
Design

The Main

For ease of tunnelling and security from water-bearing strata, and in order to avoid interference from other services, it was decided that the tunnel should be driven in London clay, wherever such formation could be met, and a suitable route was accordingly chosen. Borings were sunk at close intervals along the route to determine the level at which the main must run and hence the pressure under which it would operate. The pressure naturally varied throughout the length of the main, but the maximum pressure required a thickness of steel pipe of only $\frac{1}{4}$ inch, and a considerable length of the main needed only $\frac{3}{16}$ inch. A pipe of this thickness rolled to 75 inches internal diameter would not have kept shape and would have been unmanageable, even had the pipe-makers agreed to roll and weld it. It was therefore decided to adopt $\frac{1}{2}$-inch steel-plate pipes with 1-inch-thick spun-concrete lining, thereby providing both a stiffening and a protective coating of considerable durability. It is of course highly desirable that the periodic emptying of a main such as this, for cleaning and other maintenance purposes, must be reduced to the minimum, in view of the interruption to the service involved and the cost. A spun-concrete lining has proved invaluable in other water-supply systems in

providing a protection which needs little attention, and it was decided to adopt this lining for the main and to apply a final coating of bituminous paint. Lengths of 14 feet 6 inches (effective) were specified, each pipe being rolled from two plates with one circumferential and two longitudinal joints, all shop-welded, and the 1-inch concrete lining was to be spun inside before drawing the pipes into the tunnel. Four 3-inch-diameter

Figs 1



LONGITUDINAL SECTIONS THROUGH GROUT-HOLE SHOWING STAGES OF GROUTING OPERATION

holes with screwed steel plugs, each with a 1½-inch-diameter grout-hole and plug, were to be provided at different points in each pipe for filling the gap between pipe and tunnel-lining and final grouting if necessary (see *Figs 1*). The pipes were to be rolled with a tapering socket to enable adjoining lengths to be drawn close enough together for welding of the circumferential joint, and the concrete lining was to be stopped 3 inches clear of either end and made good by hand after welding the joint. Each joint had to

be tested at a pressure of 150 lb. per square inch, applied from inside the pipe after joint welding.

The Tunnel

The driving of the tunnel would present no trouble, being in London clay (except possibly for short sections, for which special provision was made); the main problem was the lining. For this, it was decided to adopt and develop what is known as the Don-Seg system. This system is applicable to small-diameter tunnels driven in clay, and consists of forming a ring to the exact diameter of the excavation with identical pre-cast unreinforced-concrete wedge-shaped segments. The excavation, of course, has to be done by shield. The wedges and method of operation are shown in the various illustrations accompanying the Paper. The great advantages of this system are (1) the economy effected by not using steel, since no reinforcement or bolts are required for the lining, and (2) the elimination of pressure grouting between lining and clay. Speed of erection and low cost are additional advantages, which, with the smooth internal surface obtained, fully justify the choice of this system. The leading dimensions are shown in *Figs 2* and *3*. The internal diameter of the main was 75 inches, so that, with a 1-inch concrete lining, and ½-inch steel plate, the external diameter was 78 inches, or 6 feet 6 inches. An internal diameter of 7 feet 6 inches was selected for the tunnel lining, giving an average clearance of 6 inches between the pipe and the lining, and 5 inches between the socket and lining (see *Fig. 4*). This was considered necessary for the experimental lengths since it was not known to what accuracy the tunnel lining could be placed or the pipe rolled, and the clearance required for the pipe carriage could be ascertained only after full-scale experiments had been carried out. It was hoped that the difference in diameter between pipe and tunnel-lining could be reduced still more after trials, in order to reduce to the minimum the amount of filling required in the annular space. To allow room for entering the steel pipes from the shaft, the tunnel design provided for an initial length of 23 feet 6 inches of 12-foot-diameter tunnel in standard cast-iron segments.

The Shaft

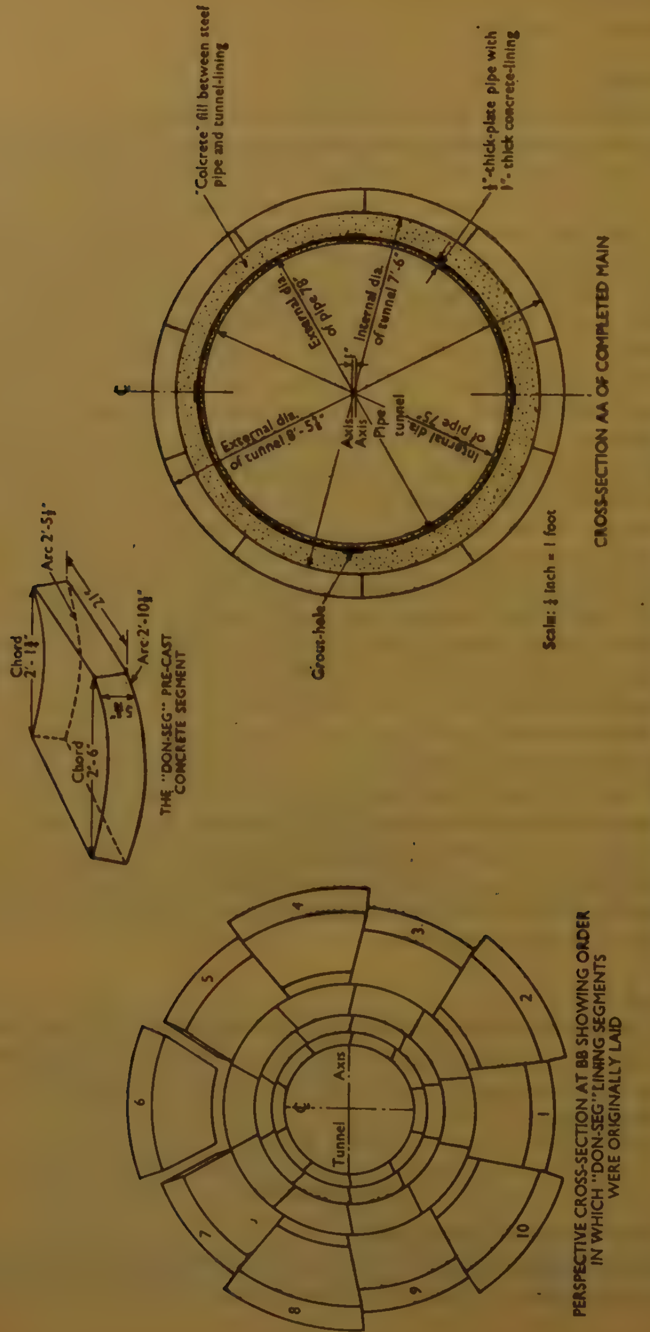
The shaft was designed to be sunk and lined with fifty-two standard 12-foot-diameter cast-iron tunnel segments, with twelve rings at the base of 16 feet 6 inches diameter. At the point chosen it was 106 feet deep to tunnel invert level.

Construction

Fabrication of Don-Seg Segments

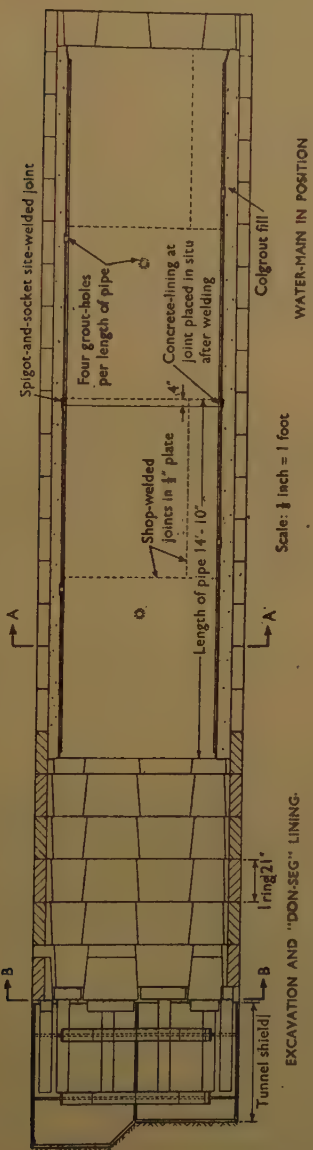
The experimental work was entrusted to Messrs Kinnear, Moodie & Co., Ltd, on a cost-plus-fee basis, and their experience in this type of work and

Figs 2



DON-SEG SEGMENTS AND MAIN DIMENSIONS OF TUNNEL

Fig. 3



LONGITUDINAL SECTION ON CENTRE-LINE OF TUNNEL

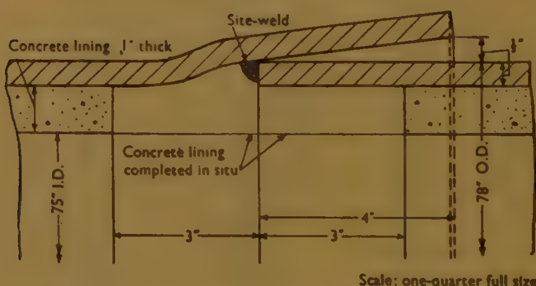
in particular in the fabrication of pre-cast concrete tunnel-lining sections was invaluable in developing successfully the hitherto untried Don-Seg segments and the system of using them in the tunnel.

The sinking of the shaft presented no problems of interest, nor did the driving and lining of the 12-foot-diameter tunnel with cast-iron linings.

Whilst this preliminary site work was in progress, the contractors cast the first of the Don-Seg concrete segments at their pre-cast concrete works. A complete ring consisted of ten segments and, for the lining to be effective, the rings had to bear tightly on the excavated clay when all segments were wedged in place with all edges flush. It was therefore essential to cast the segments with great accuracy so that, after shrinkage, they would be to the exact dimensions required.

The casting was successfully accomplished after very little trial and a master mould was made in concrete. From a segment cast in this mould,

Fig. 4



DETAIL OF SPIGOT-AND-SOCKET WELDED JOINT

further moulds were made, thus ensuring complete uniformity. The concrete mix (⅜-inch shingle, sand, and Portland cement) was 3 : 1½ : 1 by volume and the water/cement ratio was 0·42. All segments were vibrated. Tests showed crushing strengths of 4,450 to 7,250 lb. per square inch after 7 days and 6,095 to 7,590 lb. per square inch after 28 days. This was of some importance since, in the early experimental work, the segments were subjected to heavy pressure from the rams.

A smooth and uniform outside face was highly desirable, since the outside diameter is the critical one. Nevertheless, it was found impossible to cast the segments satisfactorily in a horizontal attitude with the inside uppermost, because of the concrete falling away from the cross-joints, or vertically with both inside and outside faces shuttered. The screeding of the inside face, in the former system, proved unsatisfactory, and in the latter system, too many shutter bolts were involved. It will be appreciated that, with ten segments per ring of 21 inches length, 30,170 segments per mile would be required. About 19 miles of tunnel being

involved (the remaining 3 miles being in trench), the production of more than 573,000 segments had to be envisaged. Clearly then, this was a mass-production job; any shutter design must cater for this fact and all operations such as the fastening and unfastening of bolts must be cut to the barest minimum. Finally, it was found that, with skilled workmen, a good segment could be produced by casting with the outside surface uppermost, and this system was used throughout. Subsequent development work on vertical casting, however, shows promise of proving successful and more satisfactory. With ten segments per ring, to produce a finished lining of 7 feet 6 inches internal diameter and 8 feet $5\frac{15}{16}$ inches external diameter, each segment measured $29\frac{1}{2}$ inches, tapering to $34\frac{1}{2}$ inches, with a thickness of $5\frac{15}{16}$ inches and a length of 21 inches. The segment was wedge-shaped in plan and had radial longitudinal joints designed to allow movement in two directions during the process of expanding the ring.

Great accuracy was required in casting the radial longitudinal joints. Since the segments were tapered, the radial joint had a "wind" which had to be manufactured with the greatest care to ensure perfect seating when the segments were erected.

Each of the segments weighed 3 cwt and the method of handling them at the pre-cast concrete works and when transporting to site and lowering down the shaft, required careful thought, and the contractors devised an eye-screw with a coarse thread for this purpose. The female thread was cast in one edge of the segment, no metal being used. This proved entirely satisfactory. In the tunnel, each segment could be handled by two men and lifted by four.

Tunnelling

The special tunnel shield required for the Don-Segs was designed by Messrs Kinnear, Moodie and Co., Ltd, and built by Messrs Arthur Foster (East Ham) Ltd, and was of a universal type. It had an external diameter of 8 feet 6 inches, an overall length of 5 feet 11 inches, including a 12-inch-long hood; the ten hydraulic rams which it carried each had a maximum stroke of 28 inches, the usual extension in practice being 24 inches. The shield had no tail, because the Don-Segs had to be expanded to the virgin ground, but provision was made for a tail to be studded on, should a pocket of ballast be encountered, and standard bolted segments used. It had to have ten rams, one for each segment, and was built of welded steel plates made in sections and bolted together. The shield was of very strong construction, for the pressure required on the rams was not known. To commence the drive of the tunnel, a 40-foot length was first driven and lined with standard pre-cast reinforced-concrete bolted sections grouted under pressure, this length being considered sufficient to overcome the thrust on the first Don-Seg rings while they were being expanded. Experience has

proved that this precaution will not be necessary with properly designed and cast segments.

Don-Seg Tunnel Lining

Working from the face of this lining, the first Don-Seg ring was erected and pushed home successfully, and the following thirty-eight rings were similarly erected with only one practical difficulty worth mentioning. This was a remarkable tribute to the care which the contractors had shown in perfecting the moulds for the segments and in foreseeing and forestalling the problems likely to be encountered during the erection of the rings. The one difficulty was that the completed rings proved to be a fraction of an inch too tight for the excavation. This possibility had been foreseen and, accordingly, only thirty-nine rings had been manufactured to allow of any alteration being made before the whole number of rings required were cast. The alteration was made by casting the remaining rings to a diameter $\frac{1}{16}$ inch less than the diameter of the excavation, instead of $\frac{1}{16}$ inch more as in the first rings, and all these rings closed without trouble.

The various systems tried in the erection of the early rings are of no particular interest and the following description represents the method finally adopted as being the most successful.

Excavation of the clay was carried out with two pneumatic clay-spaders, to a diameter of about 7 feet 6 inches and for a distance of 21 inches ahead of the cutting edge of the shield, leaving a 6-inch rim of clay for the cutting edge to move during the forward shove of the shield. This excavation and mucking took, on an average, approximately 90 minutes.

The rams of the shield, at this stage, were bearing on the edge of the last completed Don-Seg ring and in the retracted position. On completion of the mucking, the rams were operated by hydraulic pressure and the shield pushed forward a distance of 21 inches, leaving a clean surface of clay ready for the next ring. (See *Figs 5* and *6*.)

During the above stage, the first five segments were brought to the face on a flat bogie. On completion of the shove, any blemishes in the clay face exposed were made good with clay, though this was seldom needed, and any water from the rams was cleaned out of the invert. The segments were then man-handled from the bogie to the cleared clay for erection into a ring. Several different systems of placing and ramming home the segments were tried during the course of the work; whilst all were successful, the most satisfactory method was as follows.

Segments were numbered serially from 1 to 10, No. 1 being on the invert, No. 2 on the left, facing in the direction of tunnelling, and the remainder following in sequence clockwise, so that No. 10 came on the right of No. 1 (See *Fig. 7*).

No. 1 was placed first with the wide end leading and the narrow end 4 inches forward from the previous ring; Nos 2 and 10 were then placed with narrow ends leading and wide ends pressed against the previous

Fig. 5

Shield rams extended during shove of shield forward from erected lining

Fig. 6

View at axis level showing surface of excavation as left by shield

Fig. 8

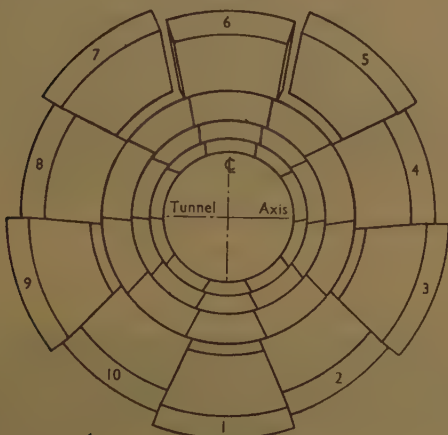
Erection of Don-Seg concrete lining, looking towards working face

Fig. 9

Ring completed, loosely stepped ready for trueing up and expansion by shield rams into arched lining completely filling the excavation

ring; Nos 3 and 9 were then placed beside Nos 2 and 10 with wide ends leading and narrow ends 4 inches forward from the previous ring. This produced one half-ring. (*Fig. 8* shows the half-ring as laid by an earlier method.) At this stage the next five segments were brought up while an erecting rib, consisting of two 9-inch-by-3-inch channels bent to form a ring 7 feet 3 inches in diameter when bolted together, was assembled just inside the last completed concrete ring. By the insertion of poling boards wedged between the erecting rib and the concrete ring, the erection of the remaining five concrete segments to form the upper half of the ring was facilitated. Each ram in the shield had two holes in its face to take steel

Fig. 7



PERSPECTIVE VIEW OF "DON-SEG" LINING SHOWING ORDER IN WHICH SEGMENTS WERE FINALLY LAID

pins to hold the leading edges of the segments, and the whole ring was then wedged up with wooden wedges against these pins. The crown segment (No. 6) was the last to be placed and, owing to the method of placing the first nine segments, which were wedged against the clay but not expanded to give the ring its full circumference, a fairly wide gap was left between Nos 5 and 7 for this segment. *Fig. 9* shows the segments laid by the earlier method.

The expanding of the ring was the next procedure and, to achieve this, the even-numbered rams (the rams being numbered to correspond with the segments) were operated to drive those segments truly home; the pressure was then released to allow the subsequent circumferential movement which would take place during the expanding of the ring. No. 1 segment was then driven hard home, then Nos 3 and 9, followed by 5 and 7. As each segment was driven home, the ring expanded, pushing the higher segments

upwards in a circular movement. This order of pushing was found to produce a tight ring with the least pressure necessary from the rams, and hence with least likelihood of damaging the concrete segments. The original segments, which had been cast to a diameter $\frac{1}{16}$ inch more than the excavation diameter, had required pressures of up to 2,000 lb. per square inch, which caused fractures at the corners of the segments. The method of expanding described above reduced the neccessary pressure to 1,000 lb. per square inch, and with segments cast to $\frac{1}{16}$ inch less than the exact diameter of excavation, pressures were reduced to 200 to 250 lb. per square inch, with a corresponding reduction in the number of fractures but no diminution of the tightness of the lining against the clay.

In the early stages the joints of the segments were liberally greased, but it was eventually found to be sufficient if they were painted with bitumastic paint prior to sending them down the shaft.

A point of some importance, in view of the small tolerance allowable between excavation and lining diameters, is that the segments must be placed immediately after the shove, otherwise the clay will expand and the segments cannot be properly closed. This fact fortunately ensures very close contact between clay and lining.

The tunnel gang consisted of one leading miner, one miner and two miner's labourers at the face, and a locomotive driver and pit-bottom man behind. On the surface the gang comprised crane driver, banksman, plant attendant, general labourer, and chainman. Two 12-hour shifts were worked, on a bonus system, for 5 days per week.

The output averaged four rings a shift, or forty rings (70 feet) a week.

No unusual difficulty was experienced in maintaining correct line and level and, when the shield did go off line, correction was easily made by the insertion of thin wood packings in the appropriate positions between concrete rings. The finished tunnel presented a smooth internal surface with no appreciable variation in diameter. The laying of decauville track on the invert, for the driving of the tunnel, was effected by using timber sleepers cut to shape at the ends. (See *Fig. 10.*)

On completion of tunnelling, the lined tunnel was cleared of all pipes, tracks, electric lights, and cables, and a 34-inch width of concrete was placed on the invert to support the 24-inch-gauge pipe-laying track, designed to give the maximum headroom.

Laying the 75-inch-Diameter Main

The 75-inch-internal-diameter concrete-lined welded-steel pipes weighed $4\frac{3}{4}$ tons each and were delivered by rail from the fabricators, The South Durham Steel and Iron Company of Middlesbrough, to London, and thence by road transport to the site. To prevent distortion and consequent cracking of the spun-concrete lining, a cruciform spider made of small-section scrap angles was welded inside each end of each pipe. The pipes were off-loaded at site by ordinary slings, and stored on specially provided

Fig. 10

VIEW ALONG TUNNEL TOWARDS WORKING FACE

Fig. 11

RECEIVING CRADLE TO SUPPORT PIPE DURING REMOVAL OF LOWERING GEAR AND BRINGING TRAVELLING GEAR THROUGH PIPE

Fig. 12

PIPE SUPPORTED ON TRAVELLING GEAR, JUST ENTERING THE 7-FOOT-6-INCH TUNNEL

Fig. 13

SMALL BOGIE IN PIPE ALREADY LAID. NEXT PIPE APPROACHING

concrete supports. The spiders were then burned out and weld scars removed by buffing. It was found that, under its own weight, the pipe distorted 1¼ inches in diameter at the spigot and ½ inch at the socket end.

A specially designed lowering gear with one pair of bogie wheels at the top end and two pairs at the bottom or landing end, was then clamped to the pipe by means of tie-rods inside and outside the pipe. The gear was fitted with lifting eyes at both ends, allowing for the pipe to be raised in a vertical position and lowered down the shaft. On arrival at the bottom of the shaft, the pipe was guided on to the decauville track and pulled to a horizontal position, with both bogies engaging the rails, by means of an air-operated winch. The pipe was then mobile, but could not be pulled long distances with this gear, which was essentially a lowering gear.

To transfer the pipe from the lowering gear to the travelling gear, an ingenious device was provided in the 12-foot-diameter section of tunnel. This consisted of slightly inclined ramps with roller wheels, so placed on either side of the track that, on drawing the pipe forward, it mounted the ramp, transferring its weight from the lowering gear to the ramp wheels. (See *Fig. 11.*) The lowering gear was then uncoupled and returned to the surface whilst the travelling gear was brought along to the pipe.

The design of the travelling gear had occasioned probably more thought than any other item of plant, since on its efficiency depended the economy of the design, which demanded the smallest possible clearance between pipe and tunnel. The gear, as finally used, consisted of a girder built from four steel channels, each of 7-inch-by-3½-inch section, supported on two A-shaped bogies fore and aft and 22 feet apart. The two legs of the A-shaped bogies could be screwed apart or together, thereby lowering or raising the girder in relation to rail level, and the extremities of the girders were bent down so that they could be carried on small bogies, thus releasing the large bogies for removal. (See *Fig. 12.*)

The A-shaped bogies were fitted with flanged steel wheels, and rubber wheels of a slightly larger diameter inside the flanged wheels. Running on poling boards laid inside the pipe, these rubber wheels carried the leading bogie (in the lowered position) up a timber ramp, through the pipe and down another ramp and back on to the tunnel track, where the flanged wheels engaged the rails. Timber saddles were then placed on top of the girders and the bogies were screwed up until the saddles took the weight of the pipe and lifted it clear of the rails. In this position it was pushed up the tunnel by an electric locomotive and was steady enough to permit a speed of 1 mile per hour.

On arrival at the last pipe laid, inside which a small independent bogie with rubber wheels was waiting, the height of the A-frame of the leading bogie was adjusted to transfer the weight on to the small bogie and to lift its own wheels clear of the laid pipe-lining. (See *Fig. 13.*) With its weight taken by the small bogie inside the laid pipe and by the large trailing bogie on the tunnel track behind, the new pipe was then pushed forward until

the spigot entered the socket of the laid pipe. A penetration of between 3 and 4 inches was usually achieved and this gave sufficiently close contact round the circumference to permit efficient welding of the joint.

By means of the trailing A-frame, the pipe was adjusted for line and level, and positioned firmly with bricks and wedges. Prior to the arrival of the pipe, the length of tunnel track which would otherwise have been covered by it was withdrawn, and the leading A-bogie of the pipe-carrying frame, after leaving the track, was carried by its rubber wheels on boards laid on the tunnel invert. Once the pipe had been secured, the pipe-carrying gear was released, and on its return journey it carried back the length of tunnel track previously removed.

Round every second pipe laid, a brick ring was built, leaving only a small hole in the soffit to permit the escape of air during subsequent grouting operations between pipe and lining.

The labour employed consisted of a tunnel gang of five with a surface gang of five, and progress averaged thirteen to fourteen pipes per week on single-shift working.

Consideration is being given to the possibility of using pipe made from much thinner plate, and some have been rolled from ¼-inch plate without concrete lining. This will make handling simpler, since the pipes will weigh only 30 cwt each, but the problem of distortion will be worse. The concrete lining will be applied mechanically after the pipe has been laid in the tunnel.

The 1-inch concrete lining showed a tendency to crack, particularly on soffit and invert, and these cracks closed as the pipe was rolled over. Most of them were only hair cracks, however, and from past experience of similar pipes, no permanent damage is anticipated, and the final coating of bitumastic paint should seal the cracks effectively.

Welding of Pipe-Joints

Each pipe-joint had to be welded to ensure watertightness, and this work was entrusted to Messrs Matthew Hall & Co. The number of runs necessary varied in general from two to four, depending upon the closeness of the joints, but where a wide gap was left (as occasionally happened), it had to be filled with electrodes as packing, and as many as six runs of welding were required. Good results were obtained and the main difficulty encountered was the fog created by the welding. Compressed air blown in did not remedy this and resort was had to extraction by fan duct. These conditions, however, are thought to be due to the fact that this particular tunnel, having only one shaft and a closed end, has no natural ventilation. In the actual work, each length of tunnel and main will have an open shaft at each end and the through draft ought to disperse all welding fog.

Testing of Welded Pipe-Joints

It is important that no water should leak into the clay in case any " backs " in the clay should permit water to reach ground surface and, even

though the pipe is surrounded by grout, every joint had to be tested for watertightness. The maximum test pressure stipulated was 150 lb. per square inch.

The design of an apparatus to test the joints from inside, which would be light enough to handle, could be collapsed to allow transfer from joint to joint, and would fit into the 6-inch gap of unlined pipe, presented many problems. It must also be realized that any piece (such as an inflatable rubber ring) which had to remain in one unbroken unit had to be threaded over any services, such as light and power cables or air lines running through the pipe, and remain so threaded until no longer required. The apparatus which was designed is in the form of a wheel which brings pressure to bear on two rubber rings, one on each side of the weld. Roughness and irregularities in the shape of the pipe create points of possible leakage, and these have to be catered for by tightening the seal with additional set-screws as required. It is felt that considerable improvements in the design may be evolved in the course of the full-scale work.

Filling between Main and Tunnel-Lining

The filling of the annular space between the steel pipes and the concrete lining of the tunnel was considered to be essential to ensure that the steel pipe should not rust, and to form a solid continuous structure of the concrete segments, the filling and the steel pipe, which could withstand both external and internal pressures and have no tendency toward any movement. For these reasons the more economical filling of blown sand was rejected in favour of concrete or grout, and the problem presented was to find the most effective method of completely filling the void with the most economical concrete mix that could be used. Strength of mix was not important.

After consideration of various alternative methods, the work was entrusted to Messrs Colcrete Ltd, who designed a special model of their standard machine to suit conditions within the main. The Colcrete machine is a combined mixing and placing machine. The cement is mixed with the water by centrifugal action; then the sand is added and mixed in by centrifugal action. The mixture is pumped through a rubber hose from the mixer to the point of placing, the whole plant being electrically operated.

The mixture used was four parts of sand to one part of Portland cement, and water was added at the rate of $7\frac{1}{4}$ gallons per cubic yard. (A further 2 gallons was assumed to be present in the sand.) The sand and cement were sent down the shaft, bagged separately in the correct proportions, and carried on a flat bogie from which the bags were transferred to a platform behind the mixer. In this way, work could continue uninterruptedly as long as required.

The Contractors provided eight 2-inch and twenty-two $1\frac{1}{4}$-inch grouting connexions for the grouting holes in the pipes, and by means of these and

suitable valves, grouting could be carried out wherever required. Colcrete filling was pumped into one pair of pipes (29-foot length) at a time, until one-third of the circumference had been filled. To avoid any chance of floating the pipes, the filling was then pumped to the next pair, the first two being completely filled on the following day.

The filling was examined by removing grout plugs from the soffit and cutting out the concrete, and in all cases the concrete examined appeared to be of very good quality, without voids.

Progress was at the rate of three and a half pipes per day in a single 12-hour day shift but it is considered that, by using special containers for the sand instead of sacks, as used in the experiment, a rate of four pipes per day could be exceeded. The gang consisted of two machine operators, two loaders, two men transferring materials from the bogie to the stage behind the machine, and one locomotive driver, a total of seven in the main. Above ground there were the crane-driver and banksman, and three labourers—a total of five men.

Pipe-Lining

After completion of the filling, joint-welding, and testing, the 6-inch gap left in the spun-concrete lining had to be filled. This work was entrusted to the Cement Gun Company, who used their " Gunite " process. Two methods were tried: in one, the gunite was sprayed directly on to the bare metal of the pipe; and in the other, a 6-inch wide strip of No. 20 XPM expanded metal was spot-welded on to the steel pipe to provide a means of adhesion for the gunite. The guniting of one joint took 15 minutes with a final coat to the soffit 2 hours later, the whole joint being given a smooth finish with trowel and brush. No signs of cracking were observed in the unreinforced joints and the reinforcement may be unnecessary.

Costs

Although careful records of costs were kept for future estimating purposes, they have not been quoted here, since without considerable adjustment to take into account the experimental nature of the work, they would be quite misleading. It may be said, however, that there is ample evidence to indicate that this system will prove highly economical.

Conclusions

The successful completion of the 1,000-foot experimental length of main in tunnel has demonstrated that the proposed design for the full-length main is not only a practical proposition, but can be built safely and rapidly, and at a cost considerably less than any other alternative method.

It is proposed to sink working shafts for the full length of main at one-mile intervals, giving a maximum drive of about ½ mile at any face. The

rates of tunnelling and pipe-laying which can be achieved show clearly that the limiting factor in programming the work will be the supply of pipes. It is for this reason and because of the steel shortage that further efforts are being made to lay the main with pipes built from steel plate only ¼ inch thick.

ACKNOWLEDGEMENTS

The Author is indebted to Mr H. F. Cronin, C.B.E., M.C., B.Sc., Vice-President I.C.E., Chief Engineer of the Metropolitan Water Board, for permission to publish the information given in the Paper and acknowledges the great ingenuity and skill shown by Messrs Kinnear, Moodie & Co., Ltd and Mr H. J. Donovan in the development of this new system of tunnelling.

The Paper is accompanied by twenty-six photographs and two sheets of drawings, from some of which the half-tone page plates and the Figures in the text have been prepared.

Paper No. 5847

"The Use of Compression Waves in Deep-Well and Borehole Pumping"

by

Eric Crewdson, M.C., B.Sc., A.M.I.C.E.

(Ordered by the Council to be published with written discussion)†

SYNOPSIS

The Author describes the development of a compression-wave pump for use in deep wells and suggests an approximate theory of its working principle.

The pump can be placed at ground level and yet can draw from a well in which the water level stands much more than 34 feet below ground. It is suitable for pumping water at a rate of up to 10 or 12 gallons per minute, from a depth below ground-level of up to 200 feet or even more. It can deliver against a positive head of as much as 100 feet or more. The foot valve is the only moving part in the well itself and is connected to the pump by a single pipe. Under suitable working conditions an efficiency of about 50 per cent can be attained.

The pump can be automatically controlled for delivering water into a pressure vessel for domestic supply.

Considerable delay in the development was occasioned by strategic mistakes in the pump's general design, and by a phenomenon of resonance which occurred in the borehole in which the pump was tested.

The development covered a period of 18 years, from 1933 to 1951.

Introduction

The problem of raising water from a depth below ground level which exceeds the height of the barometric water column has been solved in numerous ways. The principal types of deep-well pump in use at the present time are :—

(1) Pumps which have a mechanical connexion from the prime mover at ground level to the pump itself near the water surface.
(2) Submersible motor pumps.
(3) Air lift pumps.
(4) Jet pumps.

Each of these types has its appropriate field, but each also has drawbacks.

A description of a novel deep well pumping system using the compressibility of the water may therefore be of interest. The system allows the

† Correspondence on this Paper should be received at the Institution by the 1st August, 1952, and will be published in Part I of the Proceedings. Contributions should be limited to about 600 words.—Sec. I.C.E.

pump to be placed at ground level, and yet to raise water from depths below ground level greatly exceeding the barometric height. The pump is connected to the foot valve by a single pipe and the foot valve is the only working part below ground.

Working Principle

If a column of water stands in a vertical pipe, the water pressure at the bottom of the pipe is greater than the atmospheric pressure at the top by an amount equal to the static head. The water at the top is in its natural state and that at the bottom is compressed. If pressure greater than atmospheric is now applied at the top of the water column, the water throughout its length is further compressed and the pipe expanded. When this pressure is gradually released the water expands to its original volume, and the pipe retracts to its original size.

If, on the other hand, the pressure is suddenly released, the column of water expands rapidly and attains an appreciable velocity upwards along the axis of the pipe. At the moment when the water has just regained its original volume the axial velocity reaches its maximum, and before the water column comes to rest it expands beyond its original volume, with a consequent reduction of pressure at all points in the pipe from the top downwards. If suitable means of periodic application and release of pressure are provided, the absolute pressure at the lower end of the pipe periodically drops nearly to zero.

The vertical water column in the rising main is like a spring. If a cylindrical steel spiral spring is placed vertically on a table and gradually loaded it is compressed, and if the load is gradually removed it regains its original state ; during the operation its lower end remains in contact with the table. But if the load on the spring is removed suddenly the whole spring jumps upwards and its lower end rises from the table. Regarding the spring as the water column and the table as the foot valve, it is easy to see that the lower end of the water column will tend to leave the foot valve and a partial vacuum will be formed, allowing water to flow in from the well. This is the basic working principle of the pump about to be described.

Basic Elements of the Pump

The essential elements of a practical pump of this type are shown in *Fig. 1*.

A piston C is arranged to reciprocate in a cylinder D and is driven by an ordinary crank-and-axle mechanism.

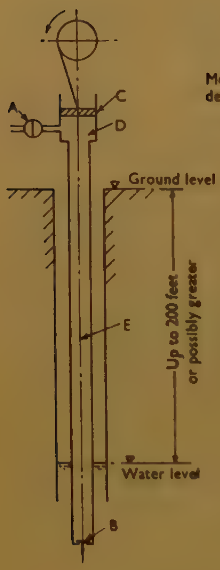
The cylinder D is connected by the drive pipe E to the foot valve B.

The mechanically operated delivery valve A is synchronized with the crank mechanism in such a way that it remains closed for a part of the time

that the piston is travelling downwards on the diagram, and remains open on the return stroke.

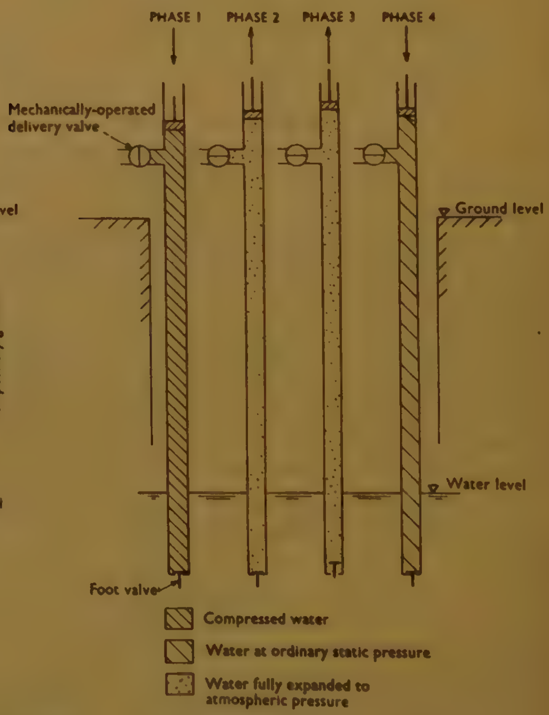
To operate the pump, the cylinder and drive pipe must first be primed with water. The motor driving the pump is then started and the piston, which is in the position shown in *Fig. 1*, compresses the water in the

Fig. 1



Basic Elements of Water-Compression Pump

Figs 2



Operation of Water-Compression Pump

cylinder and in the drive pipe, whilst the foot valve and the delivery valve are both in the closed position.

As soon as dead centre is passed, the delivery valve opens and releases the pressure in the cylinder. A wave of low pressure travels down the drive pipe to the foot valve, where the excess pressure of the water standing in the well outside the drive pipe opens the foot valve and water is allowed to flow into the drive pipe. At this instant the whole column of water is moving upwards and water is being discharged through the delivery valve.

The speed of the crank must be so arranged that by the time the piston has travelled to its extreme outer position, the upward kinetic energy of the water has been expended in work; moreover, the water in

the drive pipe must have had time to settle back to its normal volume under static pressure. At this moment the delivery valve is closed mechanically, the foot valve has already closed automatically, and the process of compression is recommenced as the piston travels downwards.

It is obvious that the depth of the water below ground level is not limited by considerations of atmospheric pressure.

The correct speed for the crank is clearly related to the length of the drive pipe; it is in fact also related to the diameter of the pipe, to the height of the water in the well above the foot valve, and to the pressure on the delivery side of the pump.

Suggested Theory of Operation

There are four phases in each complete revolution of the pump. (See *Figs 2*). In the first phase, a pressure wave is travelling down the drive pipe at a rate which may easily be calculated from orthodox water-hammer theory. This phase continues until the water in the cylinder and in the drive pipe down to the foot valve B is fully compressed. In the second phase, which begins when the delivery valve A is opened, a wave of low pressure travels down the drive pipe until it reaches the foot valve B, which then opens. In the third phase, the whole column of water in the drive pipe rises and water flows in through the foot valve and out through the delivery valve, until the upward kinetic energy of the water is expended in work. In the fourth phase, the water in the drive pipe is settling back on to the foot valve until the original static-pressure condition is regained, and the complete cycle recommences.

It seems probable that the time occupied by the first, second, and fourth phases is the same and that it is dependent upon the velocity with which the pressure wave travels along the pipe, and is independent of the compression ratio used. Since this velocity is only very slightly affected by the diameter of the pipe or its thickness, the time of these phases is proportional to the length of the drive pipe.

The time taken by the third phase depends upon the volume being delivered by the pump and is normally substantially longer than that of any of the other three phases.

No pretence is made that the above account is more than a rough outline of what, in fact, takes place in the pump, but the practical behaviour of the pump shows that no very gross error is made by adopting this simple theory if the length of the drive pipe exceeds about 100 feet. No better theory has so far been evolved.

Experimental Results

A pump with a 3-inch-diameter piston and $2\frac{1}{2}$-inch stroke was connected to a 4-inch bore vertical steel drive-pipe 140 feet long, with a foot valve

at its lower end. A vertical delivery pipe about 18 feet long was fitted to the pump discharge. The pump itself was mounted at ground level directly over an $11\frac{1}{2}$-inch-diameter borehole, and was driven by belt from an electric motor. Results of tests carried out on the 3rd and 4th April, 1946, are shown in Table 1.

TABLE 1.—TEST RESULTS

Test No.	Level of water in bore hole: feet below ground level	Revs. per minute	Delivery pressure: feet of water above ground level	Galls. per minute	B.H.P. at motor pulley	Efficiency: per cent
1	123	320	21	5·4	0·658	35·9
2	123	332	21	7·2	0·707	44·4
3	123	344	21	8·4	0·737	49·8
4	123	352	21	8·4	0·730	50·3
5	123	360	21	4·4	0·452	42·5
6	100	305	21	10·0	1·015	36·1
7	100	320	21	11·4	1·010	41·4
8	100	340	21	12·0	0·980	44·9
9	100	349	21	12·6	1·022	45·1

Theoretical comparison

The velocity of propagation of a wave of pressure in the 4-inch steel pipe used in the experiments was 4,080 feet per second. The following calculations are based on the theory put forward on p. 321.

First phase :—The time for a pressure wave to travel from the pump to the foot valve will be $\frac{140}{4{,}080} = 0{\cdot}0343$ seconds.

Second and fourth phases :—The time for each of these phases will be the same as for the first phase, that is, 0·0343 seconds.

Third phase :—To calculate this, a definite quantity of water delivered must be assumed. It would be justifiable, as a first trial, to assume that the quantity of water and speed of rotation which gave the highest efficiency are likely to give results approximating most closely to those calculated from the theory, if the theory is correct.

Test No. 4 gave the highest efficiency, and in that test 8·4 gallons per minute were delivered at 352 revolutions per minute.

If 8·4 gallons per minute were delivered, then $\frac{8{\cdot}4}{352} = 0{\cdot}02385$ gallons (6·61 cubic inches) were delivered at each pumping stroke. The amount by which the column of water rose at each pumping stroke must therefore have been :

$$\frac{6.61}{\text{area of 4-inch circle}} = 0{\cdot}525 \text{ inches.}$$

The moving column of water at the commencement of the pumping stroke (that is, immediately before the foot valve opens), is travelling upwards at a certain velocity and has a mass of

$$140 \times \frac{12{\cdot}58}{144} \times 62{\cdot}5 = 764 \text{ lb.}$$

The cross-section of the borehole is large compared with that of the drive pipe and for the purpose of this calculation may be regarded as an infinitely large reservoir.

The upward movement of the water column is restrained by its weight, together with the added pressure of a 21-foot head of water at the upper end, less the pressure of a 17-foot head above the foot valve assisting the column to rise, or a net restraining effect of a 4-foot head (1·73 lb. per square inch).

The water column is therefore decelerated by a force equal to its own weight of 764 lb. plus 1·73 lb. per square inch on the area of a 4-inch circle, that is, $1{\cdot}73 \times 12{\cdot}58 = 21{\cdot}8$ lb., which gives a total of roughly 786 lb.

The rate of deceleration of the column will therefore be $\frac{786}{764} \times 32{\cdot}2 =$ 33·1 feet per second per second.

Hence, in the usual equation of motion $S = \frac{1}{2} ft^2$, the value of S will be 0·525 inches = 0·0437 feet ; f will be 33·1 feet per second per second ; and t will be the time of the third phase in seconds.

$$\text{Hence } 0{\cdot}0437 = \tfrac{1}{2} \times 33{\cdot}1 \times t^2$$

therefore $$t = \sqrt{\frac{0{\cdot}0437 \times 2}{33{\cdot}1}} = \sqrt{0{\cdot}00264} = 0{\cdot}0514 \text{ seconds.}$$

The total time for a complete revolution is :—

1st phase, 0·0343 seconds
2nd phase, 0·0343 ,,
3rd phase, 0·0514 ,,
4th phase, 0·0343 ,,

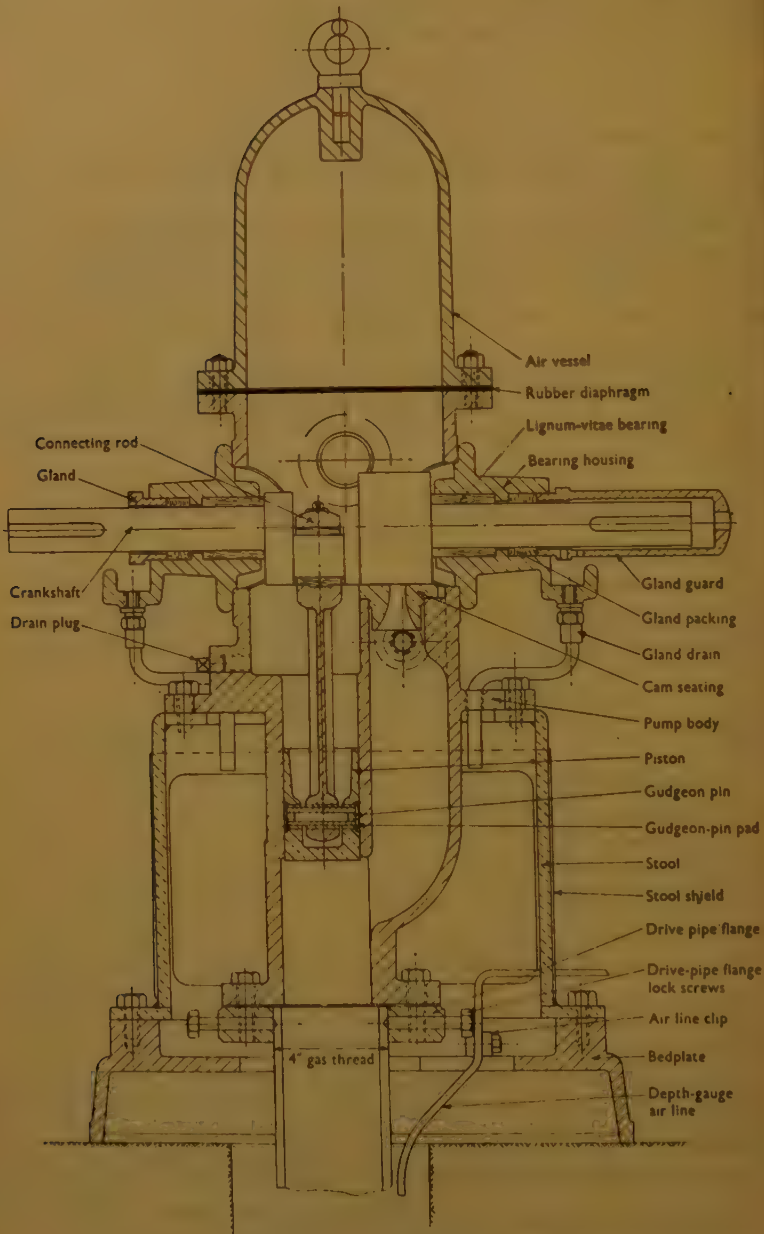
0·1543 seconds.

Hence the calculated speed $= \frac{60}{0{\cdot}1543} = 388$ revolutions per minute, or 10·5 per cent higher than the observed figure.

Similar calculations have been made for a number of observations on various sizes of pump, working with different lengths of drive pipe. The resulting figures are shown in Table 2. The figures for speed given in the last column have been calculated from a simple formula :—

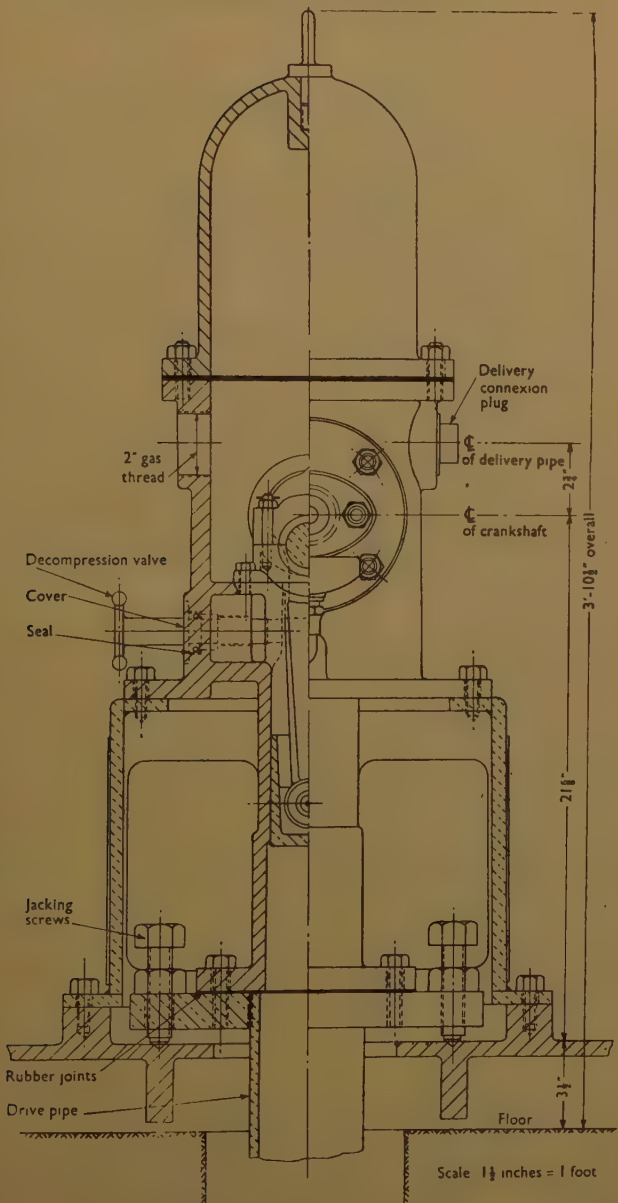
Revolutions per minute $= \frac{3{,}900}{\sqrt{L}}$ where L denotes the length of drive pipe in feet.

Figs 3 (a)



SECTIONAL VIEW OF PUMP

Figs 3 (b)



SECTIONAL VIEW OF PUMP

TABLE 2.—CALCULATION RESULTS FOR DIFFERENT PUMPS AND DEPTHS

Date	Bore : inches	Stroke : inches	Length of drive pipe : feet	Bore of drive pipe : inches	Underground head : feet	Pressure above ground : feet	Galls per minute	Measured speed : revs per minute	Speed calculated from theory : r.p.m.	Speed by formula, r.p.m. $=\frac{3,900}{\sqrt{L}}$
2 May, 1939	$1\frac{1}{2}$	$1\frac{1}{4}$	55·1	2	54·6	26·9	3·16	455	699	526
6 October, 1950	3	$1\frac{1}{8}$	70·0	3	40·0	60·0	5·5	460	647	466
17 January, 1939	$2\frac{3}{16}$	2	98·5	3	74·5	16·0	8·48	400	433	393
4 April, 1946	3	$2\frac{1}{2}$	140·0	4	123·0	21·0	8·4	352	388	330
20 July, 1950	3	$2\frac{1}{2}$	153·5	3	115·5	60·0	7·1	322	346	315
5 July, 1950	3	$2\frac{1}{2}$	253·5	3	150·5	90·0	3·93	238	246	245

This formula, so far as the Author knows, has no theoretical justification, but gives a closer approximation to the measured speeds than the theory set out above.

Practical Design

The latest design of pump tested by the Author is shown in *Figs 3* (*a*) and (*b*). The trunk piston is made of gunmetal and the body of the pump is cast iron. All the bearings except the gudgeon-pin bearings are made of lignum vitæ and designed for water lubrication. The shaft is cast iron. The gudgeon pin and its bearings are nitrided steel, and the pin is of the fully floating type. These bearings also are water-lubricated. The dome above the pump forms an air vessel. The mechanically operated delivery valve (valve A in *Fig. 1*) is formed by a cam cast on the crankshaft and so designed that its face practically closes the delivery port during a part of the downward travel of the piston, but during the rest of the revolution the cam face is quite clear (by about $\frac{3}{4}$ inch) of the delivery port. It will be clear that the pump must be driven in the correct direction of rotation, as shown on the drawing.

In the crankshaft a small hole is drilled leading from the face of the cam to the crank-pin journal. As this part of the cam face passes the delivery port, water is forced through the small hole and provides positive lubrication for the big-end bearing of the connecting rod. During the early series of experiments with this design of pump, this hole was not drilled, and the big-end bearing showed signs of fairly rapid wear. The hole was drilled to improve the lubrication and so obviate this wear, and not only was this object attained, but an unexpected effect was observed. During the whole of the previous tests, not only with this design of pump but with all the preceding designs, a certain instability of the pump had been noticed. The quantity of water delivered and the noise made by the pump had ranged periodically between fairly narrow limits at any given speed and head condition. The drilling of the hole in the cam cured this instability.

With any pump of the water-compression type it is essential that there should be no air leaks in the pump or in the drive pipe, and that there should be no possibility of air pockets forming in the pump below the delivery port.

Design of Foot Valve

The design of the foot valve is of the first importance. For the pump to operate successfully it is essential that the foot valve may open freely and promptly, and be free from leakage. The pressure surges which it has to withstand are substantially higher than the static head. The moving part of the valve is a light stainless-steel ring, which is spring-loaded and closes on to a stainless-steel annular seating. The valve

assembly can be withdrawn without disturbing the pump body or the drive piping in the borehole.

Practical Working

To test the performance of the water-compression pump under practical rather than laboratory conditions, one was sold to a firm of engineers in Hampshire who had a deep well with its water surface about 58 feet below ground level. The water from the well was used by the firm in their Works.

The pump was installed in June 1940 and has been in use since then without needing any repairs. It normally works 12 to 24 hours per week. It has been dismantled once, in 1947, so that the wear in the working parts could be measured. This was found to be small, and though some spare parts were subsequently provided they have not yet been fitted. This pump was belt-driven by a D.C. motor, and started and stopped by hand. It delivered the water only a few feet above ground level.

Automatic Working

It was felt to be essential to develop the pump for automatic starting and stopping, and for use with an air-pressure tank for normal domestic supply. Starting and stopping presented no difficulty, but to provide for make-up air in the pressure tank was not so simple. It was found that the pressure in that part of the pumping system which was above ground level was nowhere at any time below atmospheric, and a simple snifting device could not be used.

The solution adopted was the use of a water-operated injector, the bore of which was enlarged between entrance and exit by steps in the diameter. The entrance end of the injector is connected to the water space below the piston and the exit end is connected into the pump body above the piston. During the downward stroke of the piston, the pressure at the entrance is much higher than at the exit, the pressure difference being of the order of 100 to 150 lb. per square inch. During this stroke, therefore, the water flow draws air through the valve, which is of the ordinary bicycle-tire type with a special rubber tube. With the pump running at 450 revolutions per minute and lifting water 50 feet, and delivering it against 45 lb. per square inch (gauge), the injector draws in about 35 cubic inches of free air per minute. The injector delivers more air than is strictly necessary, and also slightly reduces the efficiency of the pump. It could be made smaller but the nozzle would then be so small that it might easily become clogged.

Pressure-Vessel Pumping Systems

The usual pressure-vessel pumping systems have drawbacks. In orthodox practice a pressure switch connected to the air vessel starts the

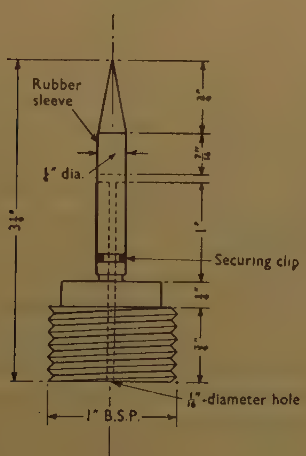
pump when the pressure falls to about 20 lb. per square inch (gauge) and stops it at about 40 lb. per square inch (gauge). Unless a water-level gauge is provided on the tank, the amount of water in the tank cannot be observed. Such a gauge is easily broken and may be a source of leakage of air. To control the water level in the tank, a float mechanism is usually installed within the tank. It is thus inaccessible and its satisfactory working cannot be readily verified. A fault in the float valve may cause the tank to be waterlogged or air to find its way into the service main.

The alternative solution, which was adopted by the Author, is to support the tank on a powerful spring and position it by means of a cylindrical steel spindle fixed to the centre of the bottom of the tank. This spindle slides in a cast-iron oil-filled housing drilled to suit the spindle, the housing being fixed with set-screws to the circular base-plate. The tank is made rather more squat than usual and forms a roof over the pump and motor. A skirt of plastic material is attached to the lower rim of the tank and reaches down to a groove cast in the base plate. The whole unit can thus be installed in the open air and the skirt protects the pump motor and switch gear from the weather.

The tank is connected by flexible piping to the pump body and to the base plate.

The vertical movement of the tank is used to operate the starting and

Fig. 4



Scale: 6 inches = 1 foot

SAFETY VALVE

stopping switch. A vertical pointer attached to the base plate outside the plastic skirt indicates on a scale on the side of the tank the amount of water in it.

The amount of water in the tank is in fact weighed—not inferred from the air pressure.

Normally the switch starts the pump when there are about 20 gallons left in the tank. If the water is drawn off at a faster rate than that at which the pump can supply, the tank continues to rise until it operates a valve in the delivery main which closes completely before the tank is empty.

To prevent the air pressure in the tank from rising too high, some form of safety-valve is necessary. The usual type of spring-controlled safety-valve tends to leak slightly under air pressure, even when that pressure is below the designed blow-off pressure. In a pressure pumping system even a slight leak of air is inadmissible.

A simple design of valve which has been found satisfactory for the purpose is shown in *Fig. 4*.

Mistakes Made during Development

In the development of any novel piece of apparatus the mistakes made are much more numerous than the successful guesses. This was certainly the case in the development of the water-compression pump. Mistakes may be divided into strategic mistakes and tactical mistakes. As in warfare, the former are the more costly.

Two strategic mistakes were made during this development.

Mistake No. 1

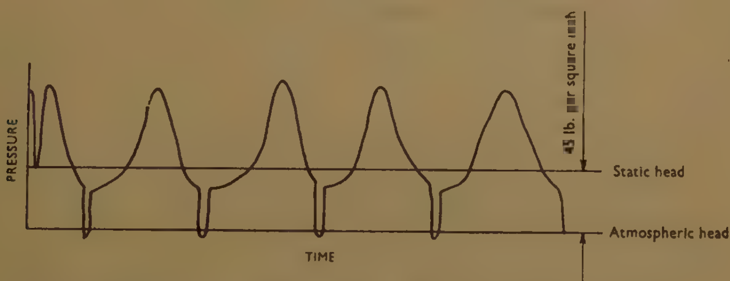
The pump which had been originally designed in 1933 as a water-compression pump was altered to work by oscillating the water bodily up and down in the drive pipe. A vessel containing compressed air was fitted immediately above the foot valve. The compression stroke of the piston forced the water as a whole down the drive pipe, and in this way further compressed the air in the air vessel. On the return stroke the water was accelerated up the drive pipe by the expansion of the air in the air vessel. One form of air vessel, which was mounted inside the drive pipe, was a rubber bag contained in a perforated metal cage so that when the rubber bag fully filled the cage no further expansion was possible. The water in the drive pipe, having attained an upward velocity, continued to move upwards when the rubber bag could not expand any more. The pressure above the foot value then suddenly fell to such an extent that water was drawn in through the foot valve. *Fig. 5* shows an indicator card of the pressure immediately above the foot valve. With this pumping system an efficiency of 60 to 70 per cent was frequently obtained, but the speed of the pump was very slow, the pump was very noisy, and there was difficulty in supplying the compressed air for the air vessel. The air vessel itself had a relatively short life.

Work on this design was carried out between 1933 and 1936, but was

ultimately abandoned after more than 900 indicator cards had been taken. In the course of this work numerous alterations in the pump itself above ground level were tried. In attempts to reduce the noise, air vessels were added here and there until the pump simply bristled with them.

After the final abandonment of the air-vessel pump, some time elapsed before the development was re-started with a water-compression pump. In the meantime, a borehole 300 feet deep and of approximately 12½ inches bore was sunk and was lined with 11¾-inch-bore steel pipe and plugged at

Fig. 5



INDICATOR CARD OF PRESSURE IMMEDIATELY ABOVE FOOT VALVE

the bottom. A hole in the ground 300 feet deep was thus made available in the Works for further experiments.

Mistake No. 2

A new water-compression pump was designed and worked reasonably well, but it was fitted with a cam-operated poppet-type delivery valve which gave a good deal of trouble. The mistake was then made of doing away with the delivery valve altogether and replacing it by what was called an "inertia pipe." This was a straight length of small-bore pipe through which the water was accelerated at a fairly high rate while the piston was working on its compression stroke. The rapid acceleration of the water in the inertia pipe implied a substantial rise of pressure below the piston during its working stroke and this rise of pressure caused a compression wave to travel down the drive pipe. As soon as the piston began its return stroke, the pressure in the pump was, of course, suddenly lowered. There was thus a similar action in the drive pipe to that in the present-day pump, and in fact the pump, once it had been got going, worked quite well and could be made reasonably quiet and efficient. It was also very simple in construction. Experiments were continued with it for 10 months during 1938. Its defects were that it was difficult to start and very sensitive to variations both of depth in the well and of pressure on the delivery side. The readings taken on this pump fill ninety-three pages

in the borehole-pump test-records book. From 1938 onwards the present design of pump was used and no more strategic mistakes were made.

The tactical mistakes were too numerous to record in detail here, but a few may be mentioned. Foot valves were the subject of many of them. Wrong materials were used for main bearings, crankshaft and small-end connecting-rod bearings. Time was wasted in making the drive pipe of different diameters throughout its length.

Natural Frequency of Water Column in Borehole

Difficulty in interpreting apparently contradictory results was experienced for years, because it was not realized that the column of water standing in the borehole had a natural frequency, varying, of course, with the length of the column.

The pump was for several years driven by a D.C. motor and the procedure in a test was to keep the water level constant and vary the speed to see what happened. Some very peculiar results were obtained in which, for nearly identical conditions, the pump delivered widely differing quantities of water. It was, moreover, sometimes found that the quantity could be greatly increased by pumping air into the water surrounding, or near to, the foot valve. But often the introduction of air made no difference at all. Then the D.C. driving motor was replaced by an A.C. one and the method of conducting the tests was altered so that tests were taken at a constant speed and the water level in the borehole was varied. It became obvious from the results obtained in this way that a critical condition existed at a depth of about 66 feet below ground level, the height of the column of water in the borehole at this stage being 234 feet.

The natural period of vibration of this water column is $\frac{4L}{a}$ seconds, where L denotes the length of the column in feet, and a denotes the velocity of propagation of a wave of pressure in feet per second. There is some doubt as to the precise value of a in the present case, because the lining tube is undoubtedly supported to some extent by the rock through which the bore is drilled. The tube is 12½ inches in outside diameter by ⅜ inch thick, which would give a value for a of 4,080 feet per second if no support was obtained from the rock. This value of a would give the 234-foot water column a natural periodicity of 262 cycles per minute, which is reasonably close to the pump speed of 280 revolutions per minute.

To bring the periodicity of the water column up to 280 cycles per minute, it would be necessary to postulate a value of 4,360 feet per second for the velocity a. This may well be the true figure. If perfect support was obtained from the rock, the theoretical value of a would be 4,675 feet per second.

Another test gave a critical speed of 268 revolutions per minute for the pump with the water surface 55 feet below ground level. The height

of the water column in the borehole was then 245 feet. If the value of a were in fact 4,360 feet per second, the natural periodicity of the water column would be 267 cycles per minute—practically identical with the actual speed of the pump.

Air pumped into the water column would, of course, radically alter its natural period of vibration and this undoubtedly is the cause of the anomalies referred to above.

Conclusion

The history of this pump's development has covered a period of some 18 years, but during the war period from 1940 to 1945 no work was done on it. It is of interest that so relatively simple a piece of apparatus should require a period of 13 years of almost continuous research for its development from the invention stage to the production of a marketable machine. The reason may lie in the fact that no wholly satisfactory theory of its working has yet been advanced, and that guesswork and trial were the only guides.

The operation of the pump has obvious analogies with that of Montgolfier's hydraulic ram. So far as the Author is aware, no complete and satisfactory theory of the hydraulic ram has ever been formulated since the ram's first use by Whitehurst in England in 1772.

Acknowledgments

The Author wishes to thank Messrs Gilbert Gilkes & Gordon Ltd. for permission to publish the experimental data given in the Paper. The original inventor of the Compression-Wave Pump was Toribio Bellocq, an Argentine engineer, whose first patent for it was taken out in 1923. It was not, however, until about 1935 that the Author had any knowledge of Bellocq's work.

The Paper is accompanied by four photographs and seven sheets of drawings, from which the Figures in the text have been prepared.

Paper No. 5868.

"Templet Method of Determining Rates of Flow and Pressures in Pipe Systems"

by

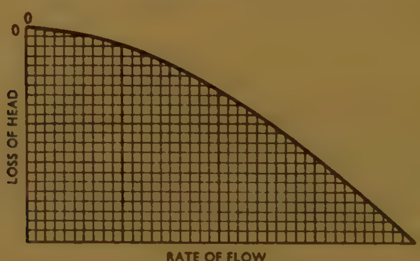
Harry Seddon, B.Sc.(Eng.), A.M.I.C.E.

(Ordered by the Council to be published in abstract form) †

The templet method of analysing the rates of flow and pressures in pipe systems devised by the Author is believed to have a rather larger range of application than the Hardy-Cross Method.

It is possible to make, either from a theoretical approach or from actual test data, a templet to give the loss of head between any two

Fig. 1



points on a main for any given rate of flow. Such a templet is shown in *Fig. 1.* Templets of this kind can be made for each section of a pipe system of network.

On a large sheet of plain or squared paper a vertical line is drawn on which can be marked to the same scale as the vertical scale of the templet all the known water levels and water pressures in the pipe system to be investigated. The templets can be fixed on this sheet by means of drawing pins. The method can perhaps be best explained by showing how the templets would be fixed to determine the rates of flow in the pipe system shown in *Fig. 2.* A possible first setting of the templets is shown in *Fig. 3.*

† The full MS. and illustrations may be seen in the Institution Library.—Sec. I.C.E.

In fixing the templets it must always be borne in mind that :—

(1) The total flow reaching any junction must equal the flow leaving it, and
(2) The head in each main at the junction is the same.

Fig. 2

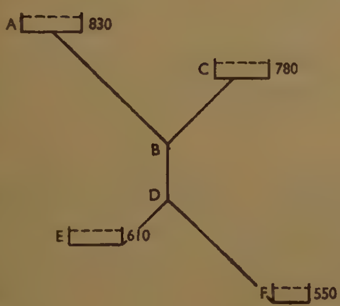
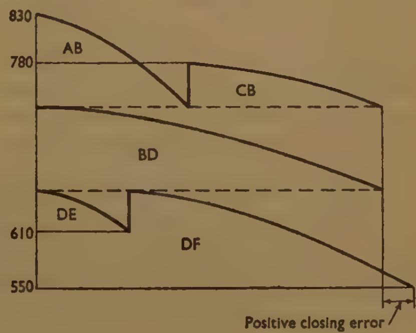


Fig. 3



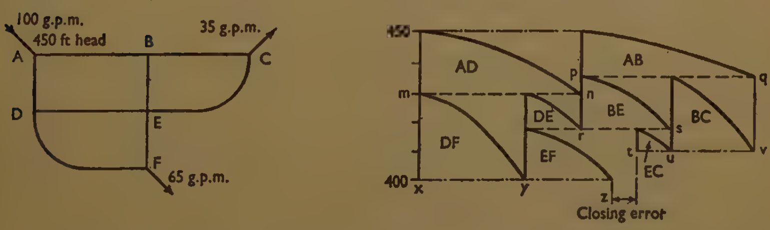
In the setting of the templets in *Fig. 3*, it will be seen that there is a positive closing error; templets AB and CB must, therefore, be re-fixed so that the combined flow is greater. The procedure is continued in this way until there is no closing error. This is usually achieved in a very few settings of the templets.

The following example illustrates the application of the method to a pipe network.

In the pipe network in *Fig. 4 (a)* it is required to find the rates of flow

Figs 4

(a) *(b)*



in the pipes and pressures at the junctions. Templets are first made for each section of the network and in *Fig. 4 (b)* the first setting of the templets is shown.

Templets AD and AB are set so that mn plus pq represents 100 gallons per minute. The flow in AB is divided between BE and BC, therefore

rs plus uv must equal pq. Point r is also the pressure at the end of DE, hence templet DE must be set to pass through point r. The pressure at F is given by the point y at which templet DF intersects the vertical axis of templet DE. The terminal pressure in EF must be the same as that in DF and is given by point z. In a similar way, templet EC must pass through the point u ; then the flow in EC is given by tu, but xy + yz + tu + uv must equal 100 gallons per minute. Therefore point t should be in the same vertical line as point z. In *Fig. 4* (*b*) this is not the case, therefore it is necessary to re-adjust the templets to give a reduced flow in AB and a correspondingly increased flow in AD. Re-adjustment is repeated until there is no closing error.

Other examples in the Paper illustrate :—

(1) The application of the method where water is pumped from a service reservoir into a distribution system, when the performance curve of the pump is known.

(2) The application to a network where there are uniform draw-offs along each section.

The Paper is accompanied by four sheets of diagrams.

OBITUARY

Sir REGINALD EDWARD STRADLING, C.B., M.C., D.Sc., Ph.D., F.R.S., who died at Shrivenham, Berkshire, on the 26th January at the age of 60, was born at Bristol on the 12th May, 1891. He was educated in the same city at the Merchant Venturers' School, the Grammar School, and the University where he graduated as a B.Sc. in civil engineering in 1912. The next 2 years he spent in practical training under the late A. P. I. Cotterell, and in gaining engineering experience with the Midland Counties Reinforced Concrete Company and with Messrs Bradshaw Gass and Hope of Bolton. In October 1914 he was commissioned in the Royal Engineers, with whom he served throughout the war. He was awarded the Military Cross and was twice mentioned in dispatches.

At the end of the war, he was appointed a lecturer in civil engineering at the University of Birmingham and while there he assisted Professor F. C. Lea in some of the latter's work as a private consultant on structural problems. In 1922 he moved to the Bradford Technical College, where he became Head of the Department of Civil Engineering, Architecture and Building, and remained there until he was appointed Director of the Building Research Station in 1924. He filled that post for 15 years, during which time he developed the Station and directed many experiments in the construction of buildings and roads. The chairman of the Building Research Board later wrote, with reference to Sir Reginald Stradling's directorship, that "the place the Station had won by 1939, and the record of its work, formed an outstanding tribute to his achievement."

He was appointed the Chief Adviser on research and experiments to the Ministry of Home Security in 1939, and Chief Scientific Adviser to the Ministry of Works in 1944. While at the Ministry of Works he was engaged upon the development of Government housing schemes, and at the end of the war he acted as adviser on civil defence to the Home Office for 3 years. Ill health obliged him to give up his full-time appointment in 1949, and he became Dean of the Military College of Science at Shrivenham.

Sir Reginald was made a Companion of the Order of the Bath in 1934 and was knighted in 1945. He played a prominent part in the work of the Institution and among his activities in this connexion it should be mentioned that he was responsible for the major part of the experimental

and theoretical investigation in the programme of research started in May 1938 by the Engineering Precautions (Air Raid) Committee of the Institution, carried out in co-operation with the Air Raid Precautions Department of the Home Office.

He was elected an Associate Member in 1917, and was transferred to the class of Member in 1928. In 1935 he was elected a Member of Council and Vice-President in 1945. He would have been nominated for the Presidential Chair had his health allowed. He was also an Associate Member of the American Society of Civil Engineers, and an Honorary Associate of the Royal Institute of British Architects. He was awarded the America Medal for Merit in 1947 for researches connected with atomic weapons. In 1943 he was elected a Fellow of the Royal Society and in the same year was awarded the Ewing Medal by the Institution of Civil Engineers.

He was married in 1918 to Inda, daughter of Mr A. W. Pippard of Yeovil. He is survived by Lady Stradling and their son and daughter.

ALBERT PLAYER ISAAC COTTERELL who died on the 16th December, 1951, at the age of 89, was born on the 26th December, 1861. He was educated at Sidcot School, Somerset, Olivers Mount School, Scarborough, and Bristol University College.

In 1878, he was articled to the late Mr F. Ashmead, who was then City Engineer of Bristol, and after the latter's death, 2 years later, he completed his training under the late Mr Francis Fox, and as assistant to the late Mr S. W. Jenkins of Liskeard. During those years he was engaged on railway extensions in Devon and Cornwall, including the laying out of the Helston railway and the design of the Cober viaduct, Redruth sewerage, Newquay water supply, and other works. In 1885 he set up in private practice as a consulting engineer in Bristol, but in 1908 he opened an office in London which became his headquarters after the 1914–18 war, the Bristol office being retained as a branch. While in practice in Bristol, he held official appointments as Surveyor to the Horfield and Barton Regis Urban District Councils, and the Chipping Sodbury Rural District Council.

He practised as a consultant, specializing in water supply and sewerage works, for 52 years and by the time he retired in 1937 he had been responsible for installations at Poole, Minehead, Bristol, Glastonbury, and elsewhere.

Mr Cotterell had many interests in addition to his practice. He was elected an Associate Member of the Institution in April 1887, and transferred to the class of Member in December 1903. He was also a Past-President of the Institution of Sanitary Engineers, a Fellow of the Royal Sanitary Institute, a Member of the Institution of Water Engineers, and a Member of the Institution of Municipal Engineers; and was at one time a member of the Bristol Corporation.

WILLIAM JONES STEELE, D.S.O., who died on the 7th October, 1951, at the age of 83, was born on the 13th September, 1868.

On completion of his training he was transferred to the staff of the Borough Engineer of West Hartlepool, the late Mr J. W. Brown, to whom he had been articled.

In 1896, he was appointed Chief Assistant to the Surveyor and Water Engineer of Tottenham, and in 1898, Deputy Chief Engineer of Bristol. Twelve years later he became City Engineer of Newcastle-on-Tyne where he remained until he retired in 1939. Duiing the 1914–18 war he served in France as an officer in the Royal Engineers and won the Distinguished Service Order.

Mr Steele, who was elected an Associate Member of the Institution in April 1899, and was transferred to the class of Member in January 1913, was Chairman of the Newcastle-on-Tyne and District Association from 1936 to 1937. He was also a Member of the Institution of Municipal Engineers.

SIR ROGER GASKELL HETHERINGTON, who died on the 24th February, was born at Highgate on the 10th February, 1876. He was educated at Highgate School and Trinity College, Cambridge, where he took the Mathematical Tripos. On leaving Cambridge he served his articles with Messrs John Taylor & Sons and Santo Crimp, for whom he was Resident Engineer on the Ilford Main Drainage Scheme and later Chief Assistant.

In 1908 he was appointed an Engineering Inspector to the Local Government Board and it was with it and its successor, the Ministry of Health, that his active career was spent.

During the 1914–18 war he was commissioned in the Royal Engineers and was Secretary to the Works Construction Sub-Committee of the Priority Committee of the War Cabinet.

He was appointed Chief Engineering Inspector to the Ministry of Health in 1930 from which post he retired in 1944, but was then retained by the Ministry as an Adviser on Water and Director of Surveys till 1947.

During his long and distinguished career in Government Service, Sir Roger was concerned with a vast number of Local Government engineering schemes and he played a very prominent part in many matters with which his Ministry was concerned. In 1929 he became Chairman of the Commission nominated by the then Home Secretary to inquire into the Holborn explosions and fires; he was a Member and subsequently Chairman (until the time of his death) of the Public Works Roads and Transport Congress and Exhibition; he served on a very large number of Government Committees and one of his most recent appointments was as a Member of the Departmental Committee appointed by the then Minister of Health to report on Greater London's Water Supplies.

He was President of the Institution of Civil Engineers for the Session

1947–48, having been elected an Associate Member in 1901; he was transferred to the class of Member in 1913 and was elected a Member of Council in 1937. He was also an Honorary Member of the Institutions of Water and Municipal Engineers.

He was awarded the O.B.E. in 1918, was made a C.B. in 1932, and was knighted in 1945.

Apart from his career at the Ministry, Sir Roger was interested in education and served for a number of years as Chairman of the Governors of Highgate School and Chairman of the Council of Wycombe Abbey School.

He was married in 1906 to Honoria, daughter of A. R. Ford, of Highgate, who survives him together with their daughter and three sons.

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